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REPORT

VOLUME I

GEOTECHNICAL INVESTIGATION LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT SANTA CLARA COUNTY, CALIFORNIA

Prepared for
Santa Clara Valley Water District
5750 Almaden Expressway
San Jose, California 95118-3686

April 28,2000

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URS

April 28, 2000
Project No. 95-973017NB.00

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Subject: Geotechnical Investigation
Lower Guadalupe River Flood Control Project
Santa Clara County, California

Dear Mr. Khan:

In accordance with Agreement No. A2091 for Professional Services (your Project No. 301511) dated November 4, 1997, URS Greiner Woodward-Clyde International-Americas, Inc. is pleased to submit this report describing the results of our geotechnical investigation for the Lower Guadalupe River Flood Control Project. This investigation includes the existing east and west levees between approximately Alviso and Highway I-880 in San Jose, California.

The accompanying report presents our opinions and recommendations regarding the geotechnical aspects of the existing levees. The opinions and recommendations are based upon the results of (1) a review of the available subsurface information, construction records and as-built drawings, (2) a site reconnaissance by walking the alignment of both levees, (3) a field investigation and laboratory testing program, (4) geotechnical analysis and (5) upon engineering judgment and our local experience.

Dr. Said Salah-Mars, Senior Project Manager, provided consultation and guidance for the earthquake engineering and slope stability aspects of the project. Mr. Mark Schmoll, Associate and Certified Engineering-Geologist, provided input for the geologic and seismic siting aspects of the project. Mr. Paul J. Boddie, G.E. 152, served as project director.

We are pleased to be of service to you on this project. If you have any questions or would like to discuss this report further, please call.

Sincerely,

S. Stephen Huang, G.E. 2150
Project Engineer

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MLL:BLW

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APPENDICES

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This report presents the results of a geotechnical investigation for the Lower Guadalupe River Flood Control project. The location and layout of the project are shown in Figure 1, Site Map. A more detailed layout of the project, including the approximate centerline of the existing west levee and east levee is shown in Figure 2, Field Exploration Plan. This figure includes the design control line with stationing presented in meters, as well as estimated limits of historical meanders, locations of existing borings and locations of field exploration points. Locations of major streets and highways crossing the Guadalupe River are also shown in Figure 2. A topographic map with aerial photography background is presented in Figures 3a through 3o; on each levee, contours (metric) are drawn with a 1-meter (3.28 feet) contour interval; stationing for the design project centerline is presented in meters. However, at the time of this report, topographic information was only available from Station 7+000 to Station 14+500. Therefore, we have not included topographic drawings north of Station 7+000 and south of Station 14+500. Throughout the text of this report, elevation, distance and stationing will be presented in metric units; except for stationing, English units will also be presented in parenthesis. However, all graphs, charts and laboratory test results are presented in English units.

The report presents the results of (1) a review of the available subsurface information, construction records and as-built drawings, (2) a site reconnaissance by walking the alignment of both levees, (3) the subsurface exploration and laboratory testing program, and (4) the results of engineering evaluations that serve as the basis for our opinions and recommendations regarding the geotechnical engineering aspects of the project.

The goal of this study is to provide a design level geotechnical investigation for the Lower Guadalupe River in order to bring the levees into compliance with the requirements of the Federal Emergency Management Agency (FEMA). Ultimately, the Santa Clara Valley Water District (District) wishes to obtain FEMA certification of the levees in order to receive a Letter of Map Revision (LOMR).

To meet FEMA standards, the levees must provide a minimum freeboard of 0.9 meter (3 feet) above the 100-year (1-percent) flood. An additional 0.3 meter (1 foot) above the minimum is required within 30.5 meters (100 feet) on either side of structures (such as bridges) riverward of the levees or wherever the flow is constricted. An additional 152 mm (1/2 foot) above the minimum at the upstream end of the levee, tapering to not less than the minimum at the downstream end of the levee, is also required. The District also must demonstrate that the levees are stable from a geotechnical standpoint.

1.1 PROJECT DESCRIPTION

The Lower Guadalupe River has been defined as that portion of the river that extends from Interstate 880 to the river's terminus with the San Francisco Bay at Alviso. This portion of the Guadalupe River is located in the City of San Jose and Santa Clara County, California. District currently anticipates that its existing levees will require raising by about 0.3 to 0.9 meter (1 to 3 feet). The extent of raising upstream of Trimble Road is expected to be less than in the downtown area.

At the south end of the project, the east levee from Interstate 880 to approximately 305 meters (1,000 feet) north of the Airport Parkway bridge is excluded from the study as a result of the Highway 87 construction. At the north end of the project, from the Union Pacific Railroad (UPRR) (formerly Southern Pacific Railroad) tracks northward, the east levee protects Alviso and the surrounding areas from flooding and ties into the existing Santa Clara County Marina parking lot. It is understood that the end of the existing levee at the County Marina is the project limit for this study. Therefore, the east levee begins at about Station 6+150 (north end) and terminates at about Station 15+300 (south end) for a total length of approximately 9,200 meters (30,185 feet). The study for the west levee will terminate at the UPRR tracks. Consequently, the west levee begins at about Station 7+000 (north end) and terminates at about Station 17+600 (south end) for a total length of approximately 10,600 meters (34,779 feet).

We have used the NGVD survey datum for the elevation of the ground surface at all existing and new borings and cone penetration tests, as well as subsurface profiles; both NGVD English and Metric units are provided in this report text. In the event that conversion to NAVD is desired in metric units, the following equation should be used:

$$\text{NAVD (metric)} = \text{NGVD (metric)} + 0.81$$

The 1-percent flood hydrograph provided by the District and developed by the Corps of Engineers is shown in Figure 4. As shown, the peak discharge is 482 cubic meters per second or cms (17,000 cubic feet per second or cfs); the duration of flow exceeding 453 cms (16,000 cfs) only lasts about 12 hours. When the discharge rate is reduced to 113 cms (4,000 cfs) or about one-quarter of the peak discharge, the time interval is about 42 hours.

1.2 SCOPE OF WORK

The following scope of work has been performed to serve as a basis for the development of the opinions and recommendations presented in this draft report:

- Reviewed available information including the following:
  - Existing subsurface information including 185 logs of borings from Woodward-Clyde and other sources. These logs were converted to gINT format and are presented in Appendix A. Existing laboratory data including consolidation and triaxial shear strength tests were also reviewed; although these laboratory tests are not shown on the logs of borings, they are presented elsewhere in Appendix B.
  - As-built drawings for the original 1963 river relocation, 1983 levee raising, and 1995 interim raising between UPRR and Highway 101.
  - Current levee cross sections and topographic drawings provided by the District.
- Interviewed District Operations and Maintenance personnel plus other project staff in a meeting on December 9, 1997 to discuss known problem areas along the project alignment. A summary of problem areas which have required maintenance or at one time seepage was reported or settlement is ongoing are presented in Appendix C.
- Selected location of new borings and cone penetration tests (CPTs).
- Cleared utilities at field exploration points through Underground Service Alert.
- Performed a detailed reconnaissance:
  - Walked the entire alignment of both levees.
  - Observed surface conditions of crest, side slopes, inboard toe and outboard toe.

  - Visually surveyed signs of observable distress such as surface cracking, wet areas, settlement, apparent deformation and undermining of toe.
  - Documented locations of observable surface stress as summarized in Appendix D.
- Performed field exploration including:
  - 34 borings to depths of 30 to 45 feet; boreholes were grouted upon completion.
  - 4 geotechnical piezometers 25 feet deep to monitor groundwater fluctuations.
  - 28 CPTs to depths of 25 to 45 feet.
- Performed laboratory testing, including:
  - Grain size distribution/hydrometer analysis
  - Plasticity index
  - Unconfined compression, water content and dry density
  - Consolidation
  - Unconsolidated undrained triaxial compression
  - Permeability
  - Direct shear
  - Consolidated undrained triaxial compression with pore pressure measurements
  - Above tests performed on both levee and foundation soils; exception is consolidation tests performed on foundation soils only.
- Performed preliminary geotechnical assessment.
- Identified key geotechnical cross-sections along both levees.
- Characterized soil profiles for these cross-sections.
- Identified potential geotechnical issues.
- Presented preliminary findings on November 10, 1998 to District staff:
  - Summarized history of geotechnical work
  - Presented results of levee characterization study
  - Discussed analysis approach and determined number of cross-sections for analysis
- Selected seven (7) most critical cross-sections.
- Performed geotechnical calculations:
  - At seven levee cross sections
  - Factors of safety for slope stability for 5 loading conditions
  - Slope stability earthquake deformations
  - Settlement
  - Seepage
  - Liquefaction potential
  - In accordance with Engineer Manual No. 1110-2-1913 (Design and Construction of Levees) and Engineer Manual No. 1110-2-1904 (Soil Mechanics Design - Settlement Analysis)
- Prepared draft geotechnical report:

- Summarized the results of all the above tasks
- Developed geotechnical opinions and recommendations regarding the structural integrity of both levees.

1.3 PREVIOUS INVESTIGATIONS

The logs of 185 previous borings completed along the levees or for projects in the immediate vicinity of the levees are provided in Appendix A. These logs represent a total of 26 different projects, over the period of 1960 through 1997. The index list for each project is shown in Table 1; a different boring number index is assigned for each project. The location of each of these borings (along with the new borings and CPTs) is shown in Figure 2, Field Exploration Location Plan. The depth of these borings ranges from 2.4 to 30.8 meters (8 to 101 feet). Most of these existing boring logs are from investigations previously performed by Woodward-Clyde. The existing borings shown on Figure 2 were selected based on these two criteria:

- They were located within or in close proximity to the footprint of the levees; and
- The technical information provided on the log was reliable, in our engineering judgment.

It should be noted that borings in the initial investigations in the early 1960s were drilled prior to construction of the levees and therefore reveal subsurface conditions only in the foundation. Subsequent borings drilled mostly in the 1980s and 1990s extended through the levee section and therefore revealed the levee fill and underlying foundation conditions at that time.

Most of the routine laboratory test data are shown on the individual boring logs, i.e. water content, dry density, and unconfined compression. The results of more complex laboratory tests such as grain size distribution, plasticity index, consolidation and consolidated undrained triaxial compression (with pore pressure measurements) from the previous investigations are presented in Appendix B.

1.4 DATA REVIEW

We have reviewed the available levee design and construction information, as well as related records, mostly provided by the District including the following:

- Drawings for the original 1963 river relocation
  - 16 sheets (some as-built) dated 1959 and 1960 and entitled “Map and General Construction Plans of Zone C-1 (Central) Project No. 3, Guadalupe River Improvement, San Francisco Bay to Branham Lane, Santa Clara County Flood Control and Water Conservation District, Santa Clara County California”; most drawings prepared by Creegan and D’Angelo.
  - 54 sheets (some as-built) dated April 26, 1963 and entitled “Map and General Construction Plans of Guadalupe River Unit 1 From Montague Road to Alviso Slough, Zone C-1 (Central), Project No. 30016, Santa Clara County Flood Control and Water Conservation District, Santa Clara County, California”; most drawings prepared by Creegan and D’Angelo.
- As-built drawings for the 1983 levee raising including 116 sheets dated September 27, 1984(as-built) and entitled “Map and Construction Plan for Guadalupe River From S.P.R.R.

to Highway 101, Central Flood Control Zone, Project No. 3015, Santa Clara Valley Water District"; logs of 10 soil borings were included on 3 plan sheets.

- As-built drawings for the 1995 interim raising between UPRR and Highway 101, including 15 sheets dated April 11, 1996 (as-built) entitled "Map and General Plan for Guadalupe River From Southern Pacific Railroad to Highway 101, Central Flood Control Zone, Project No. 3015, Santa Clara Valley Water District".
- Approximately 240 current levee cross sections.
- Current topographic and aerial photography drawings including 15 sheets dated November 25, 1997; no title; from U.P.R.R. bridge to south of Highway 101 (Airport Island Bridge). Contours are plotted on both levees using a 1.0 meter contour interval.
- Existing elevations of crest of both East and West Levees. Settlement was estimated by subtracting as-built levee crest elevations from the existing elevations.
- A plot of land subsidence from a United States Geological Survey paper entitled "Land Subsidence in the Santa Clara Valley, Alameda, San Mateo and Santa Clara Counties, California", 1971, by J.F. Poland.

We have also reviewed available District drawings and other historical records for locations of meanders. As shown on Figure 2 Field Exploration Location Plan, the approximate channel limits of old meanders are plotted for almost the entire length of the project; these are based on District drawings dated 1960 and 1963. Between Tasman Drive and Montague Expressway, the locations of earlier meanders are based on a 1873 map obtained from the archives of the San Jose Historical Museum ("Map of a Re=Survey of the "Lick" Property, Party, Part of "Rancho Rincon De Las Esteros", surveyed October 31, 1873 by J.H. Pieper and Re=Surveyed July 1887 by H.A. Braissard) were identified and plotted on Figure 2.

During construction of the levees, we understand the District typically required soft deposits within the meander and within the footprint of the levee be removed and replaced with soil spread in layers and compacted by track walking. However, no specific relative compaction requirements were set.

2.1 GEOLOGIC SETTING

The levees span from the northerly end of the Santa Clara Valley in Alviso to nearly the heart of the Valley in San Jose; the project area lies between the Santa Cruz Mountains to the west and the Diablo Range to the east. The Santa Clara Valley is a broad northwesterly trending alluvial filled basin. The older Mesozoic rocks of the Santa Cruz Mountains and the Diablo range underlie the thick accumulations of alluvial sediments in the Santa Clara Valley. These alluvial sediments are divided into the lower older Santa Clara Formation, and the upper younger surficial deposits of alluvium, and alluvial fans.

Depth to bedrock is estimated to be at least 92 meters or 300 feet (Rogers and Williams, 1974) along the levee alignment.

2.2 REGIONAL FAULTS AND SEISMIC SOURCES

The active faults that could potentially generate earthquakes producing strong ground motions along the Guadalupe River Flood Control Improvement Project are described below and are shown on Figure 5, Regional Active Faults. Each fault is characterized in terms of the type of faulting, rupture length, dip, maximum magnitude, and slip rate. The median (50th percentile) peak horizontal accelerations potentially generated by the maximum earthquake for each seismic source are tabulated in Tables 3a to 3d. The maximum magnitudes for these faults were adopted from the Working Group on Northern California Earthquake Potential (WGNCEP, 1996).

2.2.1 San Andreas Fault

The San Andreas fault extends from the Gulf of California, Mexico, to Point Delgada on the Mendocino Coast in northern California, a total distance of 1,200 km (746 miles). The San Andreas accommodates the majority of the motion between the Pacific and North American plates. This is the largest active fault in California and is responsible for the largest known earthquake in northern California, the 1906 moment magnitude (M_w) 8 San Francisco earthquake (Wallace, 1990). Movement on the San Andreas fault is right-lateral strike-slip, with a total offset of some 560 km or 348 miles (Irwin, 1990). In northern California, the San Andreas fault is clearly delineated, striking northwest, approximately parallel to the vector of plate motion between the Pacific and North American plates. Over most of its length, the San Andreas is a relatively simple, linear fault trace. However, immediately south of San Francisco Bay, the fault splits into a number of fault splays, including the Calaveras and Hayward faults. In the San Francisco Bay area the main trace of the San Andreas fault forms a linear depression along the peninsula occupied by the Crystal Springs reservoirs. Geomorphic evidence for Holocene faulting includes fault scarps in Holocene deposits, right-laterally offset streams, shutter ridges and closed linear depressions (Wallace, 1990). The 1906 earthquake resulted from rupture of the fault from San Juan Bautista north to Point Delgada, a distance of approximately 470 km (293 miles). The average amount of slip on the fault during this earthquake was 5.1 meters (16.7 feet) in the area to the north of the Golden Gate, and 2.5 meters (8.2 feet) in the Santa Cruz Mountains (WGNCEP, 1996).

Based on differences in geomorphic expression, fault geometry, paleoseismic chronology, slip rate, seismicity, and historic fault ruptures, the San Andreas fault is divided into a number of fault segments. In the San Francisco Bay Area, these segments include the Santa Cruz Mountains, possible source of the 1989 M_w 7.1 Loma Prieta earthquake and the Peninsula segment. The slip rate on these segments is 14 and 17 mm/year (0.55 and 0.67 inch/year) respectively. Both of these segments, along with the North Coast segment, ruptured during the 1906 M_w 8 San Francisco earthquake. The Santa Cruz Mountains, Peninsula, and 1906 rupture segments have calculated maximum credible earthquakes (MCEs) of M_w 7, 7, and 8, respectively (WGNCEP, 1996).

2.2.2 Hayward Fault

The Hayward fault extends for 100 km (62 miles) from the area of Mount Misery, east of San Jose, to Point Pinole on San Pablo Bay. Systematic right-lateral geomorphic offsets and creep offset of cultural features have been well documented along the entire length of the fault (e.g., Lienkaemper et al., 1991). The Hayward fault is considered to be the most likely source of the next major earthquake in the San Francisco Bay area (WGCEP, 1990). The last major earthquake on the Hayward fault, in October 1868, occurred along the southern segment of the fault. This Richter local magnitude (M_L) 7 event caused toppling of buildings in Hayward and other localities within about 5 km (3.1 miles) of the fault. The surface rupture associated with this earthquake is thought to have extended for approximately 30 km (18.6 miles), from Warm Springs to San Leandro, with a maximum reported displacement of 1 meter (3.28 feet). As well as moving during earthquake ruptures, the Hayward fault also moves by aseismic creep. Measurements along the fault over the last two decades show that the creep rate is 5 to 9 mm/year (0.20 to 0.35 inch/year) (Lienkaemper et al., 1991). The Working Group on Northern California Earthquake Potential (1996) assigns MCEs of M_w 6.9 and recurrence intervals of 210 years for both the northern and southern segments of the Hayward fault. Recent research of historical documents has led to the conclusion that an earthquake in 1836, previously thought to have occurred on the northern Hayward fault occurred elsewhere, thereby increasing the time since the last earthquake on this segment of the fault. Recent paleoseismic trenching along the northern Hayward fault indicates that the last surface rupturing earthquake along this part of the fault was sometime between 1626 and 1724 (Lienkaemper et al., 1997). This study also indicated that there are at least four surface rupturing earthquakes in the last 2,250 years.

2.2.3 Hayward Fault - Southeast Extension

The Hayward fault zone extends to the south of Milpitas, along the eastern margin of the Santa Clara Valley, as a broad zone of reverse and reverse-oblique faulting. This zone of faulting includes the Quimby, Crosley, and Evergreen faults, as well as the southeastern extension of the Hayward fault. The predominant sense of motion on these faults is right-lateral strike-slip on the continuation of the Hayward fault itself, becoming reverse-oblique, with an increasing thrust faulting component towards the west. The thrust faults dip to the northeast. Trenching investigations and the geomorphic expression of these faults indicate that they have been active during late Pleistocene, and possibly even Holocene time. The Working Group on Northern California Earthquake Potential (1996) assigns a maximum earthquake of M_w 6.5 to the southeastern extension of the Hayward fault.

2.2.4 Calaveras Fault

The Calaveras fault is a main component of the San Andreas system, branching off the main San Andreas fault south of Hollister, and extending northwards for approximately 120 km (75 miles) to die out in the area of Danville. The predominant sense of motion on the Calaveras fault is right lateral strike-slip. A smaller component of vertical displacement is evident in some areas along the fault trace. The Calaveras can be divided into two distinct sections, northern and southern, with the boundary located at Calaveras Reservoir.

The Calaveras fault has generated a number of moderate-size earthquakes in historic time, including: (1) the 1861 M_L 5.9 event; (2) the 1886 M_L 5.4 event; (3) the 1897 M_L 6.2 event; (4) a probable M_L 6.5 event in 1911; (5) the 1988 M_L 5.1 Alum Rock event; (6) the 1979 M_L 5.9 Coyote Lake event; and the 1984 M_L 6.2 Morgan Hill event.

To the south of Calaveras reservoir, microseismicity clearly delineates the active trace of the fault. The long term slip rate and contemporary creep rate for the southern Calaveras fault are approximately 15 mm/year (0.59 inch/year) (WGNCEP, 1996). The Working Group for Northern California Earthquake Potential (1996) calculated a MCE of M_w 6½ for the (southern and northern) Calaveras fault.

2.2.5 Monte Vista - Shannon Fault

The Monte Vista-Shannon fault zone is one of a series of southwest-dipping thrust and reverse-oblique faults along the southwestern margin of the Santa Clara Valley. Collectively known as the Foothills Thrust System, this is a series of range-front faults that mark the boundary between the southern Santa Cruz Mountains and the western Santa Clara Valley. The Monte Vista fault extends from Palo Alto, southeast, through Los Altos, Los Altos Hills and Cupertino to Saratoga. Rather than a single fault, this is a belt of southwest-dipping thrust faults. Limited trench data, stream channel profiles, and stream terrace elevations all indicate late Pleistocene and possibly Holocene movement on the Monte Vista thrust (Hitchcock *et al.*, 1994). To the south of Saratoga, the continuation of the Monte Vista fault is mapped as the Shannon fault. The Shannon fault also comprises a series of southwest-dipping thrust faults that offset Miocene bedrock and cut a paleosol reported to be less than 20,000-years-old [R. McLaughlin, U.S. Geological Survey, reported in Hitchcock *et al.*, (1994)]. The Monte Vista-Shannon fault is capable of generating a MCE of M_w 6¾ (WGNCEP, 1996)

2.2.6 Sargent - Berrocal Fault

The Sargent-Berrocal fault is a 94-km-long (58.4 miles), complexly branching system of southwest-dipping thrust and reverse oblique faults associated with the San Andreas fault. Evidence of recent movement along the Sargent fault include reports of surface rupture along the fault trace in the vicinity of Page Mill Road in Palo Alto during the 1906 earthquake and some possible aftershocks of the 1989 Loma Prieta earthquake. The northern two-thirds of the fault converges with the San Andreas fault at depth, and may not be an independent seismic source (WGNCEP, 1996). The Sargent-Berrocal fault is considered capable of generating a MCE of M_w 6¾.

2.3 GROUND MOTIONS

To characterize the attenuation of ground motions from the seismic sources (described above) to the Guadalupe River Flood Control Improvement Program, we have used empirical attenuation relationships appropriate for deep soil. These include Abrahamson and Silva (1997), Boore *et al.* (1993), Campbell (1997), and Sadigh *et al.* (1997). Median (50th percentile) peak horizontal ground accelerations (PGAs) were calculated for each of the designated four reaches of the Guadalupe River Flood Control Improvement Project (Tables 3a to 3d). The PGAs were calculated using the MCE for each seismic source described above, and the closest approach of each of these sources to each reach of the Project. For each site, the controlling seismic source is a M_w 8 earthquake along the 1906 segment of the San Andreas fault. This event generates median peak horizontal ground accelerations of 0.31 to 0.33 g.

2.4 SUMMARY

There are no known active faults that cross the project alignment. However, the levees will be subject to strong ground shaking from known nearby active faults.

3.1 SURFACE CONDITIONS

3.1.1 Field Reconnaissance

The surface conditions will be described on a segment by segment basis for both the east and west levees, starting at the north end at Alviso (Station 6+100) and proceeding to the south end (Station 17+600) near I 880. This description is based on our field reconnaissance during which we walked the entire alignment of both levees on December 19, 22 and 23, 1997. At that time, we observed surface conditions of the crest, both side slopes and the inboard and outboard toe of each levee. Our observations were documented in detail and are presented in Appendix D. This documentation also included areas of apparent distress such as surface cracking, wet areas, settlement, deformation and undermining of the levee toe. At the time of our field visit, the water flowing in the Guadalupe River was confined to the low flow channel; therefore, the water surface generally was below the level of the inboard toe.

3.1.2 General

In general, the east and west levees are of earthfill construction, are trapezoidal shaped in cross section and range in height from about 2 ½ to 4 ½ meters (8 to 15 feet). The crest serves as an access road and is covered with armoring aggregate (gravel). In 1995 both levees between the SPRR and Highway 101 were raised, including some areas with soil reinforcement or concrete floodwalls; in many cases the top slopes were steepened. All of the 1995 surface was surfaced with armoring aggregate; the crest width is about 4 to 5 ¾ meters (13 to 14 feet). Typically both the inboard and outboard slopes are vegetated with grass, weeds and scattered shrubs. Animal burrows are common along the entire alignment but are randomly located on the face of both side slopes. Access for vehicles is provided by the roadway on the crest and supplemented with ramps cut into the inboard slopes of both levees; at some locations an access road is also positioned just beyond the toe of the inboard slope. The low flow channel in some areas approximates the design centerline and in some areas meanders in close proximity to the inboard toe of the levee. Between the inboard toes of the levees, (i.e., in the channel area), the vegetation typically consists of weeds, grass and occasional trees.

Surface conditions for each segment are described in more detail in the following sections; refer to Appendix C for a more complete list of observations. In each segment only a few major pipes or pipeline crossings or outfalls are discussed; a more complete listing of these items is presented in Appendix C. For reference purposes for each segment, please refer to the topographic drawings in Figures 3a to 3x that include aerial photography. Problem areas identified with the District Staff in our meeting of December 9, 1997 (refer to Table 2 notes) are also noted in our discussion of each segment.

3.1.3 Segment from Santa Clara County Marina Parking Lot to UPRR Tracks-Station 6+100 to Station 7+050

Within this segment the project is limited to the east levee located in Alviso. The East Levee begins at the Santa Clara County Marina parking lot and extends southerly through Alviso to the UPRR bridge. An abandoned building (Old Cannery) that extends into the outboard slope of the

East Levee at two locations is positioned at about Station 6+270; this includes a brick retaining wall and a concrete retaining wall near the toe of the levee. Ponded water was observed (1) inside a portion of this structure on top of a concrete floor slab and (2) outside and east of this structure in a low area. Near Station 6+360 a reinforced earth wall retains the outboard slope of the levee; it appears to be in sound structural condition. An existing 2-story structure is located east of this wall. Near Station 6+180 a stockpile of dry mud was observed on the outboard slope. Within and adjacent to the river channel, numerous old boats are stored from about Station 6+200 to Station 6+800. Near Station 6+460, the Alviso Yacht Club Building is positioned on a berm extending east of the outboard slope. There are no structures abutting the outboard slope in this segment south of Station 6+460. However, the Town of Alviso lies immediately east of the levee; numerous houses and small businesses are prevalent in the area.

Scattered animal burrows were observed on both slopes throughout this segment; two clusters were noted.

3.1.4 Segment from UPRR Tracks to Highway 237-Station 7+050 to Station 8+050

Within this segment, three bridges span the river and levees at the UPRR (Station 7+050), Gold Street (Station 7+110) and Highway 237 (Station 8+050). The approach abutments of all three bridges are positioned on both levees.

3.1.4.1 East Levee

Immediately east of Gold Street, there is a widened levee section. Reportedly in 1986 or 1988 during raising of the Gold Street Bridge (refer to Location No. 8 in Table 2), asbestos was revealed in an excavation made in the vicinity of an old meander. Reportedly the asbestos was later capped with about 1.8 meters (6 feet) of silt near the outboard toe; there have been no reported post-construction problems.

A sump pump near Station 7+200 lifts water from an open ditch that collects area surface runoff; the water is pumped through a pipe that penetrates the levee.

A few minor cracks were observed on the crest between about Station 7+150 and Station 7+170; the surface was slightly depressed on the inboard edge.

A wrecking yard is located north of the levee between Gold Street and about Station 7+500. A golf driving range and golf practice fairways are located north of the levee from Station 7+500 to 7+800. A wet spot was observed near the inboard toe at about Station 7+550 and Station 7+700.

Between Station 7+800 and Highway 237, the area north of the levee is low and undeveloped. It is covered with a growth of weeds, grass and low shrubs. An apparent old water filled meander parallels the outboard toe of the levee; a tall growth of brush borders both banks of the meander. Between Station 7+690 to 7+710, minor erosion/undermining of the inboard toe was observed, as well as a wet spot.

3.1.4.2 West Levee

The area south of this levee between UPRR and Gold Street contains some small buildings, houses and an electrical transmission tower. A mobile home park abuts the levee between Gold Street and about Station 7+600.

An undeveloped area extends between Station 7+600 and Highway 237; most of the area appears to be level as a result of regrading. Two areas adjacent to the outboard toe were fenced off. One of these areas at about Station 7+630 contained standing water; miscellaneous fill consisting of chunks of asphalt concrete, portland cement concrete and soil was spread around most of the perimeter of this fence. An apparent water filled meander extends from about Station 7+800 to Highway 237; the alignment infers that it might have been part of a larger former meander that continued across both levees. Between approximately Station 7+880 and 7+910 a sag was observed for the full width of the levee crest; the adjacent fresh vegetation on side slopes and new gravel surface suggests it might have been backfill for a new pipeline.

3.1.5 Segment from Highway 237 to Tasman Drive-Station 8+050 to Station 9+250

A bridge spans the river at Tasman Drive; the approach abutments are positioned on both levees.

3.1.5.1 East Levee

Just south of Highway 237, an undeveloped area extends to about Station 8+150; within this area at Station 8+100 we observed on the outboard slope:

- an eroded gully/excavation in the upper slope; and
- a pit about 1 meter (3.3 feet) wide and 0.5 meter (1.6 feet) deep near the bottom of the slope, apparently manmade.

Beyond the toe of the outboard slope at this same location two manholes protrude above the ground surface.

From Station 8+150 to Station 8+630 a ditch parallels the outboard toe; beyond the ditch is a mobile home park. The bottom of the ditch is wet at Station 8+200 and between Stations 8+250 and 8+400. Within the mobile park is a large pond between approximately Station 8+340 to Station 8+450. At about Station 8+575 an eroded bike/foot path was observed on the outboard slope; an apparent erosion control mat had been displaced.

The Oakmead Pump Station is located beyond the outboard levee toe at Station 8+630. The outfall for this pump station consists of a concrete structure, as well as sacked concrete slope protection (SCSP) on the inboard slope of the levee. Beyond the toe and SCSP, the bottom of the channel is lined with riprap.

From Station 8+700 to Tasman Drive (Station 9+250), the adjacent property is presently under construction and appears to consist of 3-story apartments/condominiums with recreational facilities (tennis courts and swimming pool). The outboard toe of the levee was excavated to a height of about 0.5 meter (1.6 feet) and stockpiled with loose soil from about Station 8+700 to 9+150.

In close proximity to Tasman Drive (between Stations 9+200 to 9+230) the toe of the outboard slope was undermined; it appeared to be the result of an excavation for a recent pipeline installation.

Scattered animal burrows were observed along this levee.

3.1.5.2 West Levee

A storm water detention pond and pump station are positioned near Station 8+100 just beyond the toe of the outboard levee. An outfall on the inboard slope consists of three 24-inch pipes with flap valves which discharge water into the river channel.

Beginning at about Station 8+130 and continuing to Tasman Drive, a drainage ditch parallels the outboard toe at distances ranging from about 3 to 6 meters (10 to 20 feet); standing water was observed from the detention pond to about Station 8+700. Beyond the drainage ditch, the Santa Clara Golf Course is positioned on a relatively high old landfill area.

From Station 8+950 to Tasman Drive and beyond the ditch, abutting property is relatively flat and contains 1-story industrial/office buildings.

Along the levee we observed:

- A discontinuous tension crack on the upper inboard slope between Stations 8+200 to 9+150.
- Some settlement of the levee crest near the inboard hinge point between Stations 8+400 to 9+230.
- These two observations appear to be related to over-steepening of the raised fill (from 1995) along the inboard hinge point.

Just north of Tasman Drive, a construction pit for a pipeline under construction was noted.

In additional to scattered animal burrows throughout, a cluster was noted on the inboard slope near Station 9+060.

3.1.6 Segment from Tasman Drive to Montague Expressway-Station 9+250 to Station 11+820

A 6-lane wide bridge spans the river at Montague Expressway; the approach abutments are positioned on both levees.

Two buried water pipelines cross under the river at about Station 10+130. These include the San Francisco Water Department Bay Division Pipeline Numbers 3 and 4 with diameters of 1.85 meters (73 inches) and 2.32 meters (91 1/2 inches), respectively.

3.1.6.1 East Levee

With the exception of two areas, the property abutting the East Levee between Tasman Drive and Montague Expressway contains 2-story high technology buildings and parking lots.

One area of exception includes the storm-water detention pond located between Stations 10+700 and 10+950 and about 5.4 meters (18 feet) beyond the toe of the outboard levee; it is at least 2.5 meters (8.2 feet) deep. It was dry at the time of our reconnaissance. An inlet structure for the pond is positioned near Station 10+970.

Another area of exception is undeveloped and located between Stations 11+500 and 11+820 (Montague Expressway).

Along the levee, we observed:

- A concrete floodwall with an L shape cross section, is located along the outboard hinge point on the crest between approximate Stations 9+660 to 11+670
    - Several animal burrows extend under the floodwall at about Stations 9+550 and 9+880.
    - Near about Stations 10+120 and 10+200 a slight separation between the wall and backfill was noted.
    - Localized vertical hairline cracks.
- Between Stations 11+180 to 11+200 there is undermining to a height of about 0.6 meter (2 feet) at the inboard toe.
- Longitudinal tension cracks on the crest and near the crest on the outboard slope.

In addition to scattered animal burrows throughout, clusters were noted for most of this segment. In particular, numerous clusters of burrows on the outboard slope were noted between Stations 10+700 and 10+950; this coincides with the limits of the stormwater detention pond.

3.1.6.2 West Levee

The property abutting the West Levee is mostly undeveloped, with three exceptions. First, a small pond with standing water is located at about Station 9+280; it apparently controls storm flow as it contains several pipes and a concrete flow structure. Second, a storm water pump station is positioned near Station 10+160 beyond the outboard toe; 4 pipes penetrate the levee and discharge water into a concrete outfall structure at the toe of the inboard levee.

A third exception is the apartment complex located between Stations 10+900 to 11+600. These structures typically are 3 stories in height and contain a basement garage. A number of those buildings abut the outboard toe of the levee. One parking lot and several tennis courts are in close proximity to the outboard levee toe.

Along the levee we observed:

- Surface cracks from toe to crest at Station 9+750 on inboard slope.
- A steep side slope on outboard face between about Stations 9+755 to 9+850, and in particular the upper 3 meters (10 feet) between Stations 9+780 to 9+830.
- An 8-meter (26-foot) long crack at Station 10+100 on the outboard slope near the crest.
- Face of outboard slope between Station 10+400 to 10+470 was scraped off; area extended from toe to 1 meter (3.3 feet) below crest.
- A 305 mm (12-inch) apparent outfall at midheight of levee on inboard face at Station 10+600. Inlet point is unknown.
- Erosion at toe of inboard slope (apparent scour) at Station 10+900 (about 13 meters or 42 feet long) and Station 10+990 (about 20 meters or 66 feet long).
- A replaced section of levee between Stations 11+160 and 11+200 where seepage from a well had been observed in the fall of 1997. No seepage was observed during our field reconnaissance.

- A 60-meter (187-foot) long longitudinal crack between Stations 11+660 and 11+720 and another between Stations 11+745 and 11+790; both cracks are located near the top of the outboard slope.
- Two cracks on the inboard slope, one at Station 11+500 and another near Station 11+700.

3.1.7 Segment from Montague Expressway to Trimble Road-Station 11+820 to Station 13+700

3.1.7.1 East Levee

The property abutting the East Levee is developed and generally consists of 1 or 2-story high technology buildings and parking lots. An above grade water tank and associated facilities are located at Station 13+420 near Trimble Road.

Along the levee, we observed:

- Two different slopes on the inboard face between Stations 12+000 and 12+200 with a steeper apparent face at the toe. From Station 12+200 and 12+220, the toe was undercut about 0.3 meter (1 foot).
- A series of undermined local sections at the inboard toe associated with either erosion or slip failures. Maximum scarp height is 1 meter (3.3 feet) at these approximate locations.
  - Station 12+480, length 8 meters (26 feet)
  - Station 12+500, length 20 meters (66 feet)
  - Station 12+540, length 8 meters (26 feet)
  - Station 12+630, length 10 meters (33 feet)
  - Station 12+660 to 12+700
  - Station 13+000 to 13+100 (previous 1995 problem?)
  - Station 13+110, length 6 meters (20 feet)
  - Station 13+160, length 15 meters (49 feet)
- Three-meter (10-foot) long longitudinal crack at Station 13+320 near top of outboard slope.

3.1.7.2 West Levee

Along this portion of the West Levee, the abutting property has two types of developments:

- Single family residences between Station 11+840 (Montague Expressway) and 12+470; and
- Light industrial/warehouse type 1 and 2-story buildings between Stations 12+470 and 13+700 (Trimble Road), including numerous parking lots and a roadway near Station 13+300.

Along the levee we observed:

- From Station 11+950 to 12+450 there is a 1-meter (3-foot) drop off forming a shelf at the inboard toe.
- At Station 12+280 an access stairway is cut into the outboard face of the levee.

- Bulge at toe of outboard slope at about Station 12+750 and 12+860.
- Tire rutting on the gravel surface of the crest near Station 12+800.
- A 6-meter (20-foot) long longitudinal crack at Station 12+850 near top of inboard slope.

3.1.8 Segment from Trimble Road to Highway 101-Station 13+700 to Station 14+550

A 4-lane wide bridge spans the river at Trimble Road; the approach abutments are positioned on both levees. At Highway 101, an 8-lane wide bridge spans the river; its approach abutments are positioned on both levees.

3.1.8.1 East Levee

A park abuts the adjoining property from Trimble Road to about Station 14+050. Between Station 14+050 to Highway 101, the property is undeveloped and most of it is under cultivation. Several electrical transmission towers are located in this area.

Along the levee, we observed:

- Deep gully at Station 13+700 on slope at the northeast abutment of Trimble Road Bridge; it is located below the bridge drain outlet.
- Bulge at toe of inboard slope at Station 14+180 (7 meters or 23 feet long) and Station 14+200 (4 meters or 12 feet long).
- Shrinkage cracks on outboard face between Station 14+100 to 14+140 and between Station 14+200 to 14+350.
- Concrete slab serves as slope protection on inboard slope between Stations 14+350 to 14+500 (Highway 101). Two of 8 weepholes plugged.
- Scour at toe of inboard slope between Stations 14+225 to 14+330.

3.1.8.2 West Levee

The property abutting this levee is developed with light industrial buildings and parking lots from Trimble Road to about Station 14+010. From that station to Highway 101, the adjacent property is open, undeveloped and relatively flat.

Along the levee, we observed:

- Shrinkage cracks along the inboard and outboard slopes between Stations 13+800 to 14+270 and on crest between Stations 14+100 to 14+270.
- Tire ruts in gravel surface on crest between Stations 13+910 to 13+940 and between Stations 14+100 to 14+270.
- A 1,829-mm (72-inch) reinforced concrete pipe at Station 14+010 has a concrete U-type endwall outfall. Cracking was observed on the exposed invert.
- Steep inboard side slope between Stations 14+420 to 14+450.

3.1.9 Segment from Highway 101 to Highway 880-Station 14+550 to Station 15+600 ±

Within this segment there are three bridges over the Guadalupe River including one at Airport Island (2 lanes), one at Brokaw Road (5 lanes) and one at Skyport Boulevard. At each bridge the abutments are positioned on both levees.

3.1.9.1 East Levee

Between Highway 101 and Station 15+230, a large parking lot for the San Jose International Airport (SJIA) abuts the levee. From Station 15+230 the East Levee parallels Highway 87 (Guadalupe Parkway) to the end of the project at about Station 15+370.

Along the levee, we observed:

- The inboard slope underneath Highway 101 Bridge and beyond to about Station 14+580 is covered with a combination of concrete sacks and rough concrete.
- At about Station 14+660 the inboard slope above a 660 mm (26-inch) flap gate outfall is covered with concrete sacks and rough concrete adjacent to the south edge of the concrete sacks.
- The inboard slope under the east abutment of Airport Island Bridge is covered with gabion revetment.
- The levee is either small or non-existent, i.e. the channel is in a cut.
- Scour at the inboard toe between Stations 14+710 and 14+725 with a 0.3-meter (1-foot) drop.
- Between Stations 14+780 and 14+890 on inboard slope
  - old access ramp with steep side slope
  - average 1 meter (3 foot) drop at toe and a maximum drop of 1.5 meters (5 feet) at Station 14+800
- Slope change at inboard toe with a 1-meter (3-foot) drop between Stations 14+980 and 14+990.
- A 0.6-meter (2-foot) drop at inboard toe between Station 15+100 to 15+300

3.1.9.2 West Levee

Airport Boulevard parallels the west levee for the entire length of this segment; it varies in width from 2 to 4 traffic lanes. San Jose International Airport abuts this street and consists of parking lots, parking structures, terminal buildings, aircraft hangers and commercial structures. The levee in this segment is either small or non-existent or the channel is in a cut.

Along the levee, we observed:

- The inboard slope underneath Highway 101 Bridge and beyond to about Station 14+570 is covered with a combination of concrete sacks and rough concrete.
- Some of the curved inboard slope near Station 14+600 is covered with sacked concrete slope protection.
- The inboard slope under the west abutment of Airport Island Bridge is covered with a gabion revetment.

- A concrete slab covers the inboard slope from the edge of the gabion revetment to about Station 14+730. There is slight undermining of the toe of concrete slab.
- The lower part of the slope on the inboard face appears to steepen at a number of locations near Stations 14+820, 15+130, 15+360, 15+500, between 15+590 to 15+630, between 15+700 to 15+770, 15+780, between 16+890 to 17+130, between 17+340 to 17+530 (very steep face in lower 6 meters or 20 feet) and 17+570.
- A flattened inboard slope, recently regraded for the Skyport Drive Bridge now under construction, between Stations 16+160 to 16+240. Concrete abutments and bents were in place.
- Sporadic (random) concrete rubble fill (pieces and slabs) on face of inboard slope between approximately Stations 16+200 to 16+650 and 16+800 to 17+500.
- Cracking on adjacent asphalt concrete pavement on Airport Boulevard between Stations 15+090 to 15+360. However, no separation was noted between pavement and concrete curb and gutter.
- Ramps for access road cut into inboard slope with tension crack and slip between Station 15+700 to 15+770; another crack near Station 16+470.
- Steep face on inboard slope between Stations 16+890 to 17+130; possible scarp or eroded face.

3.2 SUBSURFACE CONDITIONS

3.2.1 Field Exploration

The subsurface conditions along the East and West Levees were investigated by drilling 34 exploratory borings (Borings EB-01 through EB-17 and Borings WB-01 through WB-17) and 28 cone penetration tests (CPT) (EC-01 through EC-13 and WC-01 through WC-15) at the locations shown on Figure 2. Please note that borings drilled and CPTs performed on the East Levee start with the letter E; similarly borings and CPTs on the West Levee start with the letter W. A detailed discussion of the techniques used for the subsurface investigation is presented in Appendix E. The depths of these borings and CPTs ranged from 9.1 to 13.6 meters (30 to 44-½ feet). Figure E-1 presents the Unified Soil Classification System, as well as guidelines summarizing soil consistency and relative density used in preparation of the boring logs. Figure E-2 is a Boring Log Legend which includes the notation used for the types of samples and methods of advancing them. Comprehensive descriptions of the soils encountered at each location are presented on the Logs of Borings, Figures E-3 through E-70. The soil conditions encountered are summarized in Section 3.2.3 "Soil Conditions". The CPT results are summarized in Appendix F.

3.2.2 Laboratory Testing

Soil samples were carefully sealed in the field and returned to our laboratory for testing. Soil classifications made in the field were verified in the laboratory after further examination and testing. Laboratory tests were performed on selected soil samples. These tests include, but are not limited to, water content, dry density and unconfined compressive strength. The results of these tests are presented at the corresponding sample locations on the Log of Borings. Additional tests including Atterberg Limits, sieve analysis, hydrometer, consolidation, hydraulic conductivity, and triaxial compression were made to better characterize the soil conditions and to estimate the compressibility, permeability and shear strength of the levee fill as well as the foundation material.

Six samples were selected for consolidation tests based on their quality and soil conditions encountered in the corresponding boring. A series of 16 consolidated undrained triaxial tests were conducted on 16 relatively undisturbed samples to simulate their shear strength behavior in the laboratory. Consequently, a total of 38 stress – strain plots were produced, resulting in 38 Mohr's circles. Undrained unconsolidated triaxial tests were conducted on three slightly disturbed samples to better estimate their undrained shear strengths. Three hydraulic conductivity tests were performed on three samples (two remolded and one relatively undisturbed) to estimate the permeability of sandy soils encountered along the levees. Four sieve analyses and one hydrometer test were conducted to determine the grain size distribution of selected sand and gravel samples. Thirteen Atterberg Limits tests were made on selected fine grained soils.

A more comprehensive discussion of the laboratory testing program is presented in Appendix G.

3.2.3 Soil Conditions

We have based our assessment of the soil conditions on two major sources:

- Field exploration conducted for this study (detailed logs of borings and CPTs are presented in Appendix E and Appendix F, respectively); and
- Existing subsurface data from previous investigations by our predecessor firms as well as by others (a list of the sources and the boring logs are presented in Appendix A).

For convenience of discussion, the subsurface soil and groundwater conditions along both the East and West levees are presented below with reference to the stations of the control line as shown on Figure 2. Furthermore two generalized soil profiles were developed for the East and West Levees and are presented in Figure 6, Idealized Soil Profile for East Levee and Figure 7, Idealized Soil Profile for West Levee.

East Levee

Stations 6+230 to 8+120

Levee fill extends to a depth of about 4.5 meters (15 feet). However, for the segment between Station 6+600 and 6+800, thicker dredged mud fill extending to 9.1 meters (30 feet) was encountered in previous exploration (WCC 1982). The fill consists of lean to fat clay which contains some sandy lenses and occasional wood fragments. These levee fills are very stiff to hard in consistency, conceivably due to method of placement and desiccation.

Below the levee fill, fat clay extends as deep as 11 meters (36 feet) and consists of Bay Mud with low density and high moisture content. Seams and lenses of fine sand were found throughout this layer.

Other foundation soils consist of lean clay with sand extending to 12.2 meters (40 feet). The clay is medium to stiff in consistency; it is moderately compressible. Underneath this clay layer is a sand deposit extending beyond the depth of most of the previous borings. The sand is medium dense to dense in relative density, and is relatively incompressible.

Groundwater levels measured in the borings for the past 30 years appear to reflect seasonal changes between Elevation -1.2 to 1.5 meters (-4 to 5 feet). In the current investigation, groundwater was encountered in EB-01 at approximately Elevation -2.1 meters (-7 feet).

Stations 8+120 to 9+550

Levee fill 4.6 to 5.5 meters deep (15 to 18 feet) extends generally to about Elevation 0.3 meter (1 foot). The fill consists primarily of lean to fat clay with some gravel, exhibiting very stiff to hard consistency. Below the fill, an upper lean clay extending to approximate Elevation -3.7 meters (-12 feet) is medium to stiff in consistency. Underneath this upper lean clay, a layer of lean to fat clay was found to extend to about Elevation -7.0 to -7.6 meters (-23 to -25 feet), and is soft in consistency. Below the clay strata, a deposit of dense to very dense silty sand to poorly graded sand and gravel occurs from Elevation -7.0 to -14.9 meters (-23 to -49 feet). Below the granular deposits, stiff lean clay with some sand lenses extends beyond the terminal depth of borings at about Elevation -25.3 meters (-83 feet).

Groundwater levels encountered in the borings showed variations between Elevations -1.5 and 0.9 meters (-5 and 3 feet). The fluctuation probably reflects the seasonal changes in the river levels.

Stations 9+550 to 12+450

Levee fill consisting of stiff to hard lean clay about 3.0 to 4.3 meters (10 to 14 feet) thick extends to approximately Elevation 2.4 meters (8 feet). The foundation soils consist of lean clay and interbedded sand layers extending to the terminal depths of most of the borings in this reach, at approximately Elevation -6.7 meters (-22 feet). The lean clay stratum also extends to the terminal depth of deeper boring HHP-1 at approximately Elevation -28.0 meters (-92 feet). The clay layers above Elevation -1.8 meters (-6 feet) are soft to medium in consistency and are considered to be moderately to highly compressible. Below Elevation -1.8 meters (-6 feet), the clays are medium to stiff in consistency and moderately compressible. The interbedded sand layers are generally dense to very dense with thickness varying from 0.9 to 3.4 meters (3 to 11 feet). However, in Boring GRIV-8 the sand layer is very loose in relative density.

Groundwater was not encountered in this field investigation. However, in a previous study (WCC 1992) conducted between January and February 1992, groundwater levels were noted at Elevations 0.5 to 1.5 meters (1.5 to 5 feet). It was concluded that the rise and fall of the river level had no apparent direct communication to the groundwater level encountered in the wells installed.

Stations 12+450 to 15+600

Levee fill was found to consist of lean clay with interbedded sand layers; the thickness varies from about 3.0 to 1.8 meters (10 to 6 feet) and becomes thinner towards the southern end of the channel alignment. The bottom of the levee fill varies from approximately Elevation 4.6 to 7.6 meters (15 to 25 feet). The fill material is very stiff to hard in consistency and is incompressible.

Below the fill, a stratum of lean clay with variable thickness extends to Elevations 0 ± to 6.1 meters (0 ± to 20 feet) towards the southern end of the channel alignment. This clay is medium to stiff in consistency and is considered moderately compressible. Underneath this clay stratum, dense silty sand deposits of variable thickness from 0.9 to 3.0 meters (3 to 10 feet) extend to Elevation 1.8 to 3.0 meters (-6 to 10 feet) towards the southern end of the channel alignment. Another layer of lean clay with consistency ranging from stiff to very stiff occurs below the upper sand and extends to approximately Elevation -3.7 meters (-12 feet). In deeper borings drilled previously (WCC 1982), another layer of sand and gravel was encountered to Elevation -9.8 to -12.2 meters (-32 to -40 feet). This sand deposit is very dense and is considered low in compressibility.

Groundwater levels measured in borings in this reach varied from Elevation 2.4 to 6.4 meters (8 to 21 feet), in a north to south direction. This water level appears to be consistent with the change in elevation of the upper granular layer.

West Levee

Station 6+230 to 8+500

A heterogeneous fill consisting of lean clay to silt forms the levee that extends to approximate Elevation -1.5 to 1.5 meters (-5 to 5 feet). The fill material is very stiff to hard in consistency within the upper 1.5 to 2.1 meters (5 to 7 feet) and becomes medium to soft near the fill and foundation soil interface. Between approximately Station 7+500 and 7+800, a mixture of soft, elastic silt to fat clay occurs below the levee fill and extends to approximately Elevation -9.1 to -10.4 meters (-30 to -34 feet). An old river meander occurred within this reach, so it is conceivable the silt and clay mixture are the "dredged mud" fill from the construction of the 1960 levee improvement project. At Boring WB-03, very soft, lean to fat clay was encountered between approximately Elevation -3.4 to -5.2 meters (-11 to -17 feet). The occurrence of this soft clay layer is consistent with the location of an old river meander; these deposits could be the fine sediments associated with the old river channel.

For the portion of this reach west of Station 7+500, the foundation soil below the levee fill, extending to Elevation -8.2 to -8.8 meters (-27 to -29 feet), consists of lean clay of medium to stiff consistency, and it is considered to be moderately compressible. Below the clay layer, deposits of medium dense silty sand extend beyond approximately Elevation -18.0 meters (-59 feet).

Groundwater levels measured in the 1960's exploration varied between Elevation -0.9 to 0.6 meter (-3 to 2 feet). During the current investigation, no groundwater was encountered above approximate Elevation -1.2 meters (-4 feet).

Station 8+500 to 9+200

Levee fill consisting of a lean clay and silt mixture extends to approximately Elevation 0.9 to 0 meter (3 to 0 feet). The fill material exhibits very stiff to hard consistency, and it is considered to be low in compressibility. Below the fill, a lean clay layer extends to approximately Elevation -2.4 to -3.7 meters (-8 to -12 feet). This clay layer is medium to stiff in consistency and is considered moderate in compressibility. Underneath this clay layer lies a silty sand to poorly graded sand and gravel deposit that extends beyond the terminal depths of the borings within this reach at Elevation -7.6 to -12.2 meters (-25 to -40 feet).

Groundwater levels measured in the 1960's and 1980's investigations varied between Elevation -0.3 to 0± meters (-1 and 0± feet). Boring WB-04 of this investigation revealed no groundwater above Elevation 0.9 meter (3 feet).

Station 9+200 to 10+300

Levee fill extends to approximately Elevation 1.5 to 3.4 meters (5 to 11 feet). The fill consists of a lean to fat clay mixture of stiff to very stiff consistency; it is considered to be low in compressibility. Underneath the fill, in general a layer of fat clay overlies a lean clay sublayer and clayey sand to poorly graded sand and gravel deposit. The fat clay extends to approximately Elevation -8.5 to -4.6 meters (-28 to -15 feet) towards the southern end of this reach; it is soft to medium in consistency. The lean clay sublayer occurs towards the southern end of the reach and is stiff in consistency. The granular deposits underneath the clays are medium dense to very dense and extend to approximately Elevation -13.7 to -15.2 meters (-45 to -50 feet). Deeper borings such as TDB-2 and HHP-4 revealed a lean clay deposit extending beyond Elevation -24.4 meters (-80 feet); the clay soil was medium to stiff in consistency.

Groundwater levels measured in the 1960's and 1980's investigations varied from Elevation 0 to -0.3 meter (0 to -1 foot). Boring WB-05 for this investigation encountered no groundwater above Elevation 0.8 meter (2½ feet).

Station 10+300 to 11+750

Levee fill varies from 4.0 to 5.2 meters (13 to 17 feet) in thickness in this reach and extends to approximately Elevation 1.5 to 3.0 meters (5 to 10 feet). The fill is stiff to hard in consistency in the upper 3.0 meters (10 feet); it becomes medium near the fill and foundation soil interface.

Underneath the fill lies a deposit of lean clay, with a sublayer of fat clay towards the northern end of the reach; it is a continuation from the preceding reach. The lean clay layer is medium to stiff in consistency, whereas the fat clay sublayer is soft to medium. The bottom of this clay layer varies between approximately Elevation -2.4 to -5.2 meters (-8 and -17 feet). A 0.6 to 0.9 meter (2 to 3-foot) thick, loose to medium dense silty sand layer occurs below the clay layer.

Groundwater levels measured in the 1960's investigation varied between approximately Elevation 2.1 and 3.7 meters (7 and 12 feet). Boring WB-06 for this investigation encountered no groundwater above approximately Elevation 2.1 meters (7 feet).

Station 11+750 to 14+000

Levee fill 4.6 to 5.5 meters (15 to 18 feet) in thickness extends to about Elevation 3.0 to 7.6 meters (10 to 25 feet); this fill thins to about 2.4 meters (8 feet) south of Trimble Road. The fill is stiff to hard in consistency in the upper 3.0 meters (10 feet); it becomes medium to stiff below approximately Elevation 4.6 meters (15 feet). Below the fill lies a layer of lean clay where the bottom extends to approximate Elevation -1.5 to 4.6 meters (-5 to 15 feet); this clay is medium to stiff in consistency. A deposit of silty sand underlies the clay; the sand is medium dense to dense. Below the sand is a medium to very stiff lean clay extending to approximately Elevation -4.9 meters (-16 feet).

First groundwater encountered during field exploration varied between approximately Elevation 4.0 and 4.6 meters (13 and 15 feet). Stabilized groundwater was measured at approximately Elevation 5.8 meters (19 feet).

Station 14+000 to 15+550

Levee fill consisting primarily of lean clay extends to approximately Elevation 8.2 to 10.1 meters (27 to 33 feet). The clayey fill is very stiff to hard in consistency. Underneath the fill is a layer of lean clay which extends to approximately Elevation 6.1 to 3.0 meters (20 to 10 feet). The clay is soft to medium in consistency. Underlying the clay is a sand to silty sand deposit extending to approximately Elevation 1.5 to 4.6 meters (5 to 15 feet); the sandy deposit is medium dense. Another lean clay layer occurs below the sandy deposit and extends beyond the terminal depths of borings within this reach at approximately Elevation -1.5 meters (-5 feet). The clay is medium in consistency.

Groundwater levels measured in the 1980s varied between Elevation 5.5 and 6.7 meters (18 and 22 feet); the stabilized water level was at about Elevation 9.5 meters (31 feet) in Boring SJIA-2. No groundwater level was encountered above Elevation 7.3 meters (24 feet) in Boring WB-14.

Station 15+550 to 15+800

About 3.0 meters (10 feet) of levee fill was encountered to approximately Elevation 10.7 meters (35 feet). The fill consists of very stiff lean clay. The foundation soils consist of highly variable interbedded layers of clays and sands. In the south half of this segment, a 2.1 to 4.3-meter (7 to 14-foot) thick layer of stiff to hard fat clay underlies the fill. A stiff lean clay layer underlies the fill and fat clay and extends to between approximately Elevation 6.1 and 3.0 meters (20 and 10 feet). It is considered to be moderate in compressibility. About 0.9 to 3.7 meters (3 to 12 feet) of silty sand underlies the lean clay and extends to approximately Elevation 0.6 to 3.0 meters (2 to 10 feet). The sand is medium dense. A soft to stiff lean clay layer underlies the sand and extends to approximately Elevation -5.5 to 1.2 meters (-18 to 4 feet). Another deposit of sand occurs below the lean clay and extends to approximately Elevation -4.6 to -9.4 meters (-15 to -31 feet); the sand is dense to very dense. Underneath the sand deposit is a lean clay layer that extends beyond the terminal depth of Boring SJIA-21 in this reach to approximately Elevation -12.8 meters (-42 feet).

Groundwater levels measured in Borings WB-15 and SJIA-S2 varied between Elevation 7.0 and 7.3 meters (23 and 24 feet). Stabilized groundwater levels measured in Boring SJIA-16 and SJIA-18 varied between Elevation 9.1 and 9.5 meters (30 and 31 feet).

Station 15+800 to 16+750

Levee fill about 2.4 to 3.0 meters (8 to 10 feet) thick extends to approximately Elevation 12.2 to 12.8 meters (40 to 42 feet). The fill consists of a lean and fat clay mixture stiff to very stiff in consistency. Underneath the fill, a layer of fat clay occurs to about Elevation 7.9 to 8.5 meters (26 to 28 feet); it is medium to stiff in consistency. Directly below the fat clay layer, foundation soils consist of a layer of lean clay and an underlying sand layer that extends beyond the terminal depths of borings within this reach. The sand stratum is at least 2.4 to 4.3 meters (8 to 14 feet) thick. The lean clays are soft to stiff in consistency, whereas the sand deposits are loose to medium dense.

Groundwater levels measured in borings within this reach varied between approximately Elevation 8.8 and 10.1 meters (29 and 33 feet).

Station 16+750 to 17+600

Levee fill about 1.5 to 3.0 meters (5 to 10 feet) thick overlies lean clay strata with an interbed of medium dense silty sand. The fill extends to approximately Elevation 13.7 to 16.8 meters (45 to 55 feet) and is very stiff to hard. The lean clay deposit underneath the fill extends beyond the terminal depths of the borings within this reach; it is very stiff to hard in consistency to a depth of about 5.5 meters (18 feet) and is considered moderate in compressibility. Below this deposit, the lean clay layer is soft to medium. The silty sand interbed is about 0.6 to 1.5 meters (2 to 5 feet) thick and is positioned at about Elevation 12.5 to 13.1 meters (40 to 43 feet).

Groundwater levels measured in nearby Boring WB-16 became stable at approximately Elevation 9.0 meters (29½ feet). At Boring WB-17, no groundwater level was encountered above Elevation 12.2 meters (40 feet).

4.1 CROSS SECTIONS SELECTED FOR ANALYSIS

During our meeting of November 10, 1998, we presented two color-coded subsurface profiles, one for the east levee (Figure 6) and a second for the west levee (Figure 7). Our interpretation of the soil layers in each profile was based on our evaluation of the previous and new subsurface information, including both borings and cone penetration tests (CPT). Based on these two profiles, we identified a total of 23 levee sections as candidates for analysis; the sections are listed in Table 4. At your request, we then selected what we believe represent the seven (7) most critical sections of the 23; a list of these seven (7) sections is presented in Table 5. The basis for selection of the seven (7) sections is presented below.

4.1.1 Selection Of 23 Levee Cross Sections

The two idealized soil profiles (West Levee profile and East Levee profile) presented in our November 10, 1998 meeting were based on plotting the logs of 185 previous borings, as well as 34 new borings and 28 new CPTs. Based on our engineering judgment and interpretation of these two profiles, we identified 13 levee sections along the west levee (Sections W1 through W13), and 10 levee sections along the east levee (Sections E1 through E10). The approximate channel control line station limits of each section are listed in Table 4. Although a total of 23 levee sections for both West and East Levees were initially identified, in our opinion three (3) sections of the West Levee (Sections W2, W9 and W10) have similar subsurface conditions as the East Levee (Sections E2, E9 and E10, respectively). Therefore, from an engineering viewpoint, we believe the subsurface conditions along the project alignment can be represented by a reduced total of 20 levee sections.

4.1.2 Selection of Seven Levee Cross Sections

According to our Agreement, a maximum of seven (7) levee cross sections were to be analyzed. As your staff requested in our meeting of November 10 1998, we have selected seven (7) sections from the 23 listed in Table 4, which we believe are reasonably representative of the most critical conditions. Critical is defined as most likely having the lowest factor of safety for slope stability, based on our engineering judgment. The seven (7) most critical levee sections we selected are shown in Table 5 and are defined as Sections 1 through 7. The reasons for selecting these seven (7) are listed under the column entitled "comments" in Table 5; they vary from the presence of soft, fat clay (CH) to very loose sand in the levee foundation, to steep side slopes and a ditch at the outboard toe. More specifically, levee sections were also selected on geometric considerations including:

- W3 and W4 based on drainage ditch at toe of outboard slope
- W7 with apparent steep side slopes
- W12 with apparent steep side slopes

We prepared two drawings for each of the seven (7) sections: (1) one drawing showing a cross section with a boring log representing the soil and existing geometric profile, and a table summarizing the selected geotechnical design parameters (Figures 8 through 14); and (2) a second drawing showing four graphs where the undrained shear strength (S_u), moisture content, dry density and blows per foot are plotted with respect to elevation (Figures 15 through 21). The latter drawing includes data from all available borings within the Station limits of the selected section. As shown in Figure 16 for example, the data for Section 2 includes that from four (4) borings (GRIV-15, GRIV-16, EB-08, and WB-03) and one (1) CPT (WC-03). The District has furnished all geometric levee sections.

4.1.3 Selection of Shear Strength Parameters

At each of the seven levee cross sections, shear strength parameters were selected for each soil layer based on the field and laboratory data. The selected design parameters are summarized in the upper left-hand corner table of Figures 8 through 14 for Sections 1 through 7, respectively. These methods are described in more detail as follows.

Cohesive Soils

Undrained Strength

As an example, refer to Section 1 on Figures 8 and 15 where three soil layers are shown including:

- Layer 1 is lean to Fat Clay fill (CH), about 3.7 meters (12 feet) thick
- Layer 2 is Fat Clay (CH), about 9.5 meters (31 feet) thick
- Layer 3 is Sand (SP), about 0.6 meter (2 feet) thick

As shown in S_u versus elevation in Figure 15, all of the unconfined compression, undrained unconsolidated triaxial shear strength tests (UU) and CPT tests results are plotted. Based on this plot for Layer 1, a vertical S_u line (Elevation 4.9± 0.9 meter or Elevation 16 to 3± feet) of 120 kPa [2,500 pounds per square foot (psf)] was selected. For Layer 2, a vertical S_u line (Elevation 0.9 to -8.5 meters or Elevation 3 to -28± feet) of 28.7 kPa (kilo Pascals) (600 psf) was selected. These values were then entered in the table at the upper left-hand corner of Figure 8. It should be noted that the S_u value selected is not a lower bound; in some cases the S_u values plotted are believed to underestimate the soil strength because of the presence of sand interbeds in the tested samples.

In a similar manner, the moisture content and dry density for each layer in Section 1 was obtained from the other plots in Figure 15 and entered into the table in the upper left hand corner of Figure 8.

Effective Strength

The data from consolidated undrained (CU) triaxial shear strength tests with pore pressure measurements is presented in Table 6. These test results include both existing tests and new tests performed for this investigation and divided into the following five categories:

- Lean clay fill (CL), dry density < 1,762 kilograms per cubic meter (kcm) [110 pounds per cubic foot (pcf)]

- Lean clay fill (CL), dry density > 1,762 kcm (110 pcf)
- Fat clay (CH), dry density < 1,281 kcm (80 pcf)
- Lean clay (CL), 1,281 kcm (80 pcf) < dry density < 1,602 kcm (100 pcf)
- Lean clay (CL), 1,602 kcm (100 pcf) < dry density < 1,762 kcm (110 pcf)

Two effective strength parameters are shown in Table 6 including ϕ' (angle of internal friction) and c' (cohesion); values are listed for peak stress, 10 percent and 20 percent strain criteria. A design value was selected between the peak stress and 10 percent strain; in our judgment 20 percent strain is too large a deformation for the levee to undergo. The value from Table 6 was then entered into the upper left-hand corner table of Figure 8 for Layers 1 and 2. For example, Layer 1 was lean clay fill with a dry density of 1,762 kcm (110 pcf); from Table 6 for 1,762 kcm (110 pcf), $\phi' = 33$ degrees and c' = 0 kPa (0 psf) were selected. These values were also compared to a CU test result for nearby boring EB-01 where $\phi' = 27$ degrees and c' = 11 k Pa (230 psf) were measured. It was our engineering opinion values of $\phi' = 33$ degrees and c' = 0 kPa (0 psf) were more representative of Layer 1; therefore these values were shown for the line entitled "selected".

Table 6 was prepared based on the graphical CU test results plotted in Figures 22 through 26. For example for the lean clay fill category with samples with dry density less than 1,762 kcm (110 pcf) for peak stress, all q versus p values were plotted in Figure 22. The q and p values are defined as follows:

- $q = \frac{1}{2}(\sigma_1' - \sigma_3')$
- $p = \frac{1}{2}(\sigma_1' + \sigma_3')$

The best fit straight line (method of least squares) was then plotted in Figure 22 for peak stress and the ϕ' and c' values calculated (as shown in that figure) as 23.7 degrees and 21.8 kPa (456 psf), respectively.

Effect of Creep of Fat Clay Fill

As a result of the review of our draft report dated July 14, 1999, the District has asked if the shear strength parameters used in the slope stability analysis for highly plastic, potentially expansive clays (i.e. fat clays defined as CH) should be reduced for creep at about 15 percent strain. Our response is as follows.

First, the longitudinal length of levees with CH fill is relatively limited, i.e. of order 10 percent of total levee length. In particular CH materials are prevalent as follows:

- East Levee
    - From about Station 6+230 to 7+990
    - About 1,760 m (5,773 feet) in length
    - Alviso area to north of Highway 237
    - For full height of levee
    - Includes our Borings EB-01, EB-02 and EB-03

  - Moisture content ranges from 13 to 22 percent with average of 17 percent
  - Cross Section 1 shown in Figure 8
- West Levee
  - From about Station 7 + 000 to 7 + 915
  - About 915 m (3,000 feet) in length
  - Between Gold Street and Highway 237
  - Varies from about 1/3 to full height of levee
  - Includes our boring WB-01
  - Moisture content ranges from 19 to 26 percent with average of 22 1/2 percent.

During our field reconnaissance in December 1997, we observed no shrinkage cracks on the side slopes (inboard and outboard) of the area just described.

Second, none of the fill samples in the above listed borings that we reviewed revealed slickensides. In our judgment, measured moisture contents of the fill samples were either close to or exceeded inferred optimum moisture content. Therefore, the fill samples were compacted in a plastic state, not a brittle state. Therefore, creep along slickensides is unlikely.

Third, our CU triaxial shear strength test results for compacted CH fill indicate effective strength parameters within the anticipated range of overburden stress (maximum of 76.6 kPa or 1,600 psf) will probably result in a higher factor of safety against slope stability. One set of circles was performed for CH fill; this sample was at a depth of 2.7 to 3.5 meters (9 to 11.5 feet) in Boring EB-01. If shear strength parameters at 15 percent had been used (c'=35.9kPa or 750 psf and ϕ'=10.6 degrees), in our judgement the high c' value would have distorted the results of our stability analysis and resulted in higher factors of safety than those reported. This would have been misleading.

Fourth, we have reviewed geotechnical literature and found no basis for selecting shear strength at 15 percent strain for CH soils. Some references (Bureau of Reclamation, 1974 and Leonards, 1962) mention that the test should be continued to 15 or 20 percent strain, and that failure can be defined at maximum deviator stress.

In summary, although CH materials were encountered in limited fill areas, it appears the likelihood of shallow surface slides due to creep is relatively small; this is particularly the case for the West Levee where CH fill was only a portion of the embankment. Although the East Levee contained CH fill for full height, no cracking was observed on the side slopes. Nevertheless, as a precautionary measure, we recommend that additional surface soil samples (below topsoil) be obtained and tested from both side slopes between Stations 6+230 to 7+ 990 on the East Levee; Atterberg Limit tests should be performed to determine if they are CL or CH materials. If these shallow materials are CH, they could be removed to a depth of about 600 mm (2 feet) and replaced with engineered fill classified as CL. Alternatively, the CH soils could be chemically stabilized with lime to lower the plasticity. Both of these remediation methods are consistent with EM 1110-2-1913, Section 6-8 “Surface Slides”.

Granular Soils

Effective strength parameters were estimated by the following method (refer to Layer 3, sand, in the upper left-hand corner table of Figure 8). The effective stress σ_v' at the middle of Layer 3 was calculated to be about 71.8 kPa (1,500 psf). For a Standard Penetration Test N value of 25 and σ_v' of 71.8 kPa (1,500 psf), the estimated relative density (D_r) is about 80 percent (see Figure 27). According to a published graph (see Figure 28) by Schmertmann (1975) for a rounded sand and D_r of 80 percent, ϕ' is about 36 degrees.

In conclusion, it is our opinion the seven sections selected and listed in Table 5 (Sections 1 through 7) are reasonably representative of the subsurface conditions that may be critical from a levee stability and design viewpoint. We performed slope stability analyses at each of these sections based on the parameters summarized in Figures 8 through 14 for the existing levee geometry.

4.2 METHOD OF ANALYSIS

Five design conditions were analyzed for the slope stability evaluation in accordance with the Army Corps of Engineers Manual for the Design and Construction of Levees (EM 1110-2-1913). The conditions, the slopes analyzed, shear strength guidelines, and minimum factors of safety required for design are listed below.

Case Number	Design Condition	Slope Analyzed	Minimum Factor of Safety
I	End of Construction	Riverside and Landside	1.3
II	Sudden Drawdown	Riverside	1.0
III	Critical Flood Stage	Riverside	1.4
IV	Steady Seepage at Flood Stage	Landside	1.4
VI	Earthquake	Riverside and Landside	1.0

The stability of the levees was evaluated using a limit equilibrium method based on Spencer's procedure of slices as coded in the program UTEXAS3 (Wright, 1990). In Spencer's procedure, all forces are assumed to have the same inclination, and all requirements for static equilibrium are satisfied. The trial and error solution involves successive assumptions for the factor of safety and side force inclination until both force and moment equilibrium are satisfied.

In general we used the circular arc method in the search for the critical failure surface. However, a wedge failure analysis was performed for Section 3. In this section, it was believed that a wedge failure could occur in the soft clay layer (Layer 3) near the surface of the foundation. UTEXAS3 was also used to search for the most critical wedge failure surface. The wedge analysis was performed for the sudden drawdown case that had resulted in the lowest factors of safety with the circular search.

The SEEPW program (Geo-Slope International, 1992) was used to estimate the pore pressure contours for four of the critical levee sections based on maximum 100-year storm water levels provided by the District. The SEEPW program is described in further detail in Section 6 of this report.

A hand calculation was performed to verify that the results generated by the UTEXAS3 program were reasonable. The Finite Slice Modified Swedish Method was used. This method assumes that the interslice forces act parallel to the average outside slope angle. The procedure outlined in the Army Corps of Engineers Manual for the Stability of Earth and Rock-Fill Dams (EM 1110-2-1902), dated April 1970 was followed. The most critical failure produced by the UTEXAS3 computer program was for Section 2 during sudden drawdown.

4.2.1 Design Hydrograph

In order to assess the advancement of the wetted face into the levee during high flow stages, we developed a time rate of change of the river level (at a given cross section) based on the design hydrograph in Figure 4. This hydrograph was discussed previously in Section 1.1 "Project Description". For a given cross section and design condition, the wetted face (phreatic surface) was estimated. Based on this phreatic surface, we then estimated the pore pressures in the levee for Case Numbers II through IV.

4.2.2 Static Loading Conditions

The static loading conditions for slope stability analysis prescribed by the Army Corps of Engineers are listed below.

- End of Construction
- Sudden Drawdown
- Critical Flood Stage
- Steady Seepage

A discussion of the conditions for analysis and the strength parameters is found in the following sections.

End of Construction

The end of construction case represents the condition immediately following construction when excess pore pressures have been induced by the construction, and not enough time has passed for them to dissipate. For this reason, undrained conditions exist for an impervious embankment and foundation soils. Strength results from laboratory unconsolidated-undrained tests were used for the fine grained soils and the effective friction angle was used for the granular soils.

Sudden Drawdown

The sudden drawdown case represents the condition whereby a prolonged flood stage saturates at least the major part of the upstream (riverside) embankment portion and then drops faster than the soil can drain. This causes the development of excess pore pressures that may result in the upstream slope becoming unstable.

The sudden drawdown case was analyzed with three-stage computations using the UTEXAS3 computer program. For the first stage, the drained strengths were used in conjunction with the water surface at the critical flood stage with full seepage. In the second and third cases, a nonlinear shear strength envelope was used for the fine grained materials. The strength envelope consisted of the undrained strengths until the intersection of the undrained strength was reached. At that point and beyond, the drained strength was used. This strength envelope is in accordance with that specified by the Army Corps of Engineers Manual.

Critical Flood Stage

The critical flood stage case represents the condition whereby an intermediate prolonged flood stage saturates the embankment and a condition of steady seepage is established. The design shear strength of impervious soils should correspond to a strength envelope midway between the R and S envelopes where the S strength is less than the R strength. The design strength of granular soils should correspond to the S envelope. Consistent with the Army Corps of Engineers Manual EM 1110-2-1902, the R and S tests are defined as follows. Tests corresponding to these drainage conditions are (1) R tests in which consolidation or swelling is allowed under initial stress conditions but the water content is kept constant during application of shearing stresses, and (2) S tests in which full consolidation or swelling is permitted under the initial stress conditions and also for each increment of load during shear.

For this analysis, a composite envelope was used for the design strengths for impervious soils. Drained strengths were used until they became greater than the undrained strengths. At this point and beyond, an envelope midway between the drained and undrained strengths was used. These strengths are in accordance with the Army Corps of Engineers Manual.

Steady Seepage From Full Flood Stage

The steady seepage case represents the condition where the water remains at or near full flood stage so that the embankment becomes fully saturated and a condition of steady seepage occurs. Design shear strengths and water levels for this condition are the same as described for the critical flood stage.

4.2.3 Earthquake Loading Conditions

Earthquake

The earthquake case represents the condition when the levee is subjected to earthquake motion. The Army Corps of Engineers Manual states that this case may be analyzed for the end of construction or critical flood stage conditions. For this analysis, the stability for the end of construction case was determined. An analysis of the earthquake case for the critical flood condition was not performed because of the low probability of a major earthquake coinciding with the 72-hour duration of the 100-year storm event.

The value of the seismic coefficient was assumed to equal the value of the peak horizontal ground acceleration in g's since the levee height is relatively small. The peak ground acceleration was determined using a deterministic approach where no consideration is given to the probability of an earthquake occurring. The maximum earthquake magnitudes adopted by the Working Group on Northern California Earthquake Potential (1996) were used for each of the seismic sources. Empirical attenuation relationships were used to calculate the median horizontal ground accelerations as discussed previously in Section 2.3 "Ground Motions". Accelerations were determined for four segments along the reach of the Lower Guadalupe River. A summary of the sections and their corresponding maximum accelerations is presented below.

Section	1	2	3	4	5	6	7
Acceleration	0.31g	0.31g	0.31g	0.31g	0.32g	0.32g	0.33g

The strength parameters used for the earthquake condition were the same as for the end of construction case.

4.3 STABILITY RESULTS

The minimum (critical) factors of safety computed with the various methods of analysis are summarized in the table below.

	Factor of Safety							
Design Condition	Minimum Required	Section 1 Minimum	Section 2 Minimum	Section 3 Minimum	Section 4 Minimum	Section 5 Minimum	Section 6 Minimum	Section 7 Minimum
I-End of Construction	1.3	3.6	3.3	3.9	4.2	4.0	6.8	10.2
II-Sudden Drawdown	1.0	1.1	1.0	0.9	2.6	2.5	1.0	2.7
III-Critical Flood Stage	1.4	1.5	1.8	3.4	4.0 *	6.1 *	2.4	3.4 *
IV-Steady Seepage at Flood Stage	1.4	1.6	1.3	1.3	2.3	5.8	1.7	11.5
VI-Earthquake	1.0	1.2	1.2	1.3	1.2	1.3	2.1	4.4

* In these 3 locations the factor of safety is based on manual hand calculations.

The approximate locations of the critical slip surfaces are shown on Figures 8 through 14.

The analyzed sections, with the exception of Sections 2 and 3, appear to be stable for each of the design conditions. The outboard slope of Section 2 was found to be problematic in the steady seepage at flood stage condition where the estimated factor of safety (1.3) was less than the minimum factor of safety (1.4) required for design. Both the inboard and outboard slopes of

Section 3 were found to be problematic in the sudden drawdown and steady seepage conditions where the estimated factor of safety (0.9 and 1.3, respectively) was less than the minimum factor of safety (1.0 and 1.4, respectively) required for design.

The wedge analysis performed for Section 3 produced a factor of safety above 2.0 for the worst-case sudden drawdown condition. This factor of safety exceeds the minimum required. At other locations (sections) weak clay layers are not usually found near the original ground surface. Therefore, in our engineering judgment we believe a wedge-type of failure is unlikely to occur along the levee alignments.

The factor of safety estimated by hand calculation using the Modified Swedish Method was found to be less than the required factor of safety for design for a saturated levee condition. Since the levees are composed of more impervious clay materials, the factor of safety was re-calculated for a moist levee condition. The factor of safety for the moist condition was slightly above that calculated with the computer program. The UTEXAS3 computer program is judged to produce results comparable to hand solutions.

5.1 SUBSIDENCE

From 1916 to 1966 in the San Jose area of Santa Clara Valley, the artesian head declined from 55 to 73 meters (180 to 240 feet) in response to groundwater withdrawal. As a direct result of the artesian head decline, the land surface subsided as much as 3.9 meters (12.7 feet) in San Jose (Poland and Ireland, 1985). The recovery of artesian head since 1967 has been substantial. In downtown San Jose, the artesian head recovered 21.3 to 30.5 meters (70 to 100 feet) in the 8 years to 1975. The subsidence has been stopped by raising the artesian head in the aquifers. From 1934 to 1960, maximum subsidence of 1.5 meters (5 feet) occurred in San Jose and Sunnyvale. From 1960 to 1967, maximum subsidence of 1.1 meters (3-1/2 feet) occurred in San Jose and Santa Clara; the average rate of subsidence was 152 mm (1/2 foot) per year, the most rapid rate of subsidence in any 7-year period. Based on the land subsidence contour map prepared by United States Geological Survey (Poland and Ireland, 1967), the segment of Guadalupe River within the scope of study experienced subsidence ranging from 1.8 to 2.4 meters (6 to 8 feet) between the period of 1934 and 1967. Since 1967, subsidence has been significantly reduced. In San Jose Wells 7S/1E-16C6 and C11, an extensometer recorded compression decreasing from 305 mm (1 foot) in 1961 and 1962 to 76 mm (1/4 foot) in 1967 and to 3 mm (1/100) foot in 1973. Based on the referenced contour map, a maximum drop of 1.1 meters (3½ feet) in elevation of the levees since their improvement construction in 1960 can be accounted for by land subsidence.

5.2 LEVEE SETTLEMENT

5.2.1 Inferred Settlement

Based on review of the (1) as-built drawings for the 1960 and 1963 improvement construction of the levees and (2) those for the 1985 raising and widening of the Guadalupe River channel, we have estimated the levee settlement between Station 7+000 and Station 14+500. Estimates of the settlement are inferred because they are primarily based on the change in elevation of the crest of the levees. Our review indicates that the lowering of crest elevations occurred mostly in areas north of Station 12+000 (Montague Expressway). The levee portions south of Station 12+000 appear to have experienced little or no change in crest elevations. We estimate that the portions of levees north of Station 12+000 settled due to consolidation approximately 229 to 762 mm (average of 381 mm) or ¾ to 2½ feet (average of 1¼ feet) for the West Levee and 396 to 1189 mm (average of 933 mm) or 1¼ to 4 feet (average of 3 feet) for the East Levee for the period of 1960 to 1983. These inferred consolidation settlements deduct the subsidence settlement estimated from 1960 to 1967. North of Station 12 +000, we estimated the subsidence settlement to be 457 to 762 mm (1½ to 2½ feet) during this 7-year period.

5.2.2 Predicted Settlement

These estimates have been compared to settlement calculations based on consolidation test results from 1960 studies and our current investigation (1998). It is assumed that the height of the fill at each stage of construction was added instantaneously. The time rate of settlement for the seven sections are presented graphically on Figures 29 to 35. The following table summarizes the calculated settlements from 1960 to 1997.

Section	Estimated Consolidation Settlement					
	From 1960 to 1983		From 1963 to 1995		From 1963 to 1999	
	Mm	inches	mm	inches	mm	inches
1	1168	46	1702	67	1715	67½
2	400	15¾	1080	42½	1187	46¾
3	743	29¼	1111	43¾	1314	51¾
4	559	22	648	25½	692	27¼
5	718	28¼	743	29¼	870	34¼
6	19	¾	19	¾	19	¾
7	102	4	102	4	102	4

Sections 1 through 5 are located at or north of Montague Expressway. The values shown in the above table are consistent with those estimated (inferred) from the as-built drawings. For example, the maximum estimated consolidation settlement from the above table at Section 1 is 1,168 mm for the period 1960 to 1983. As described in Section 5.2.1 "Inferred Settlement" the upper bound of inferred consolidation settlement was 1189 mm. Therefore, we have concluded the approach and methodology used in our settlement analysis are appropriate. Also shown in Figures 29 through 35 are the accumulated settlement estimated for the period from 1983 to 1999.

5.3 FUTURE SETTLEMENT

For preliminary assessment purposes, we have assumed that future raising of the levees would be approximately 0.9 meter (3 feet). We have estimated long term consolidation settlement for the seven sections and they are presented below:

Section	**Incremental Estimated Future Settlement Due to 0.9 meter (3 feet) of New Fill**	
	mm	inches
1	203	8
2	171	6¾
3	267	10½
4	38	1½
5	235	9¼
6	6	¼
7	19	¾

Therefore, the future settlement due to 0.9 meter (3 feet) of new fill placed on the levees generally north of Montague Expressway is estimated to be of the order of 254 mm (10 inches) and south of Montague Expressway is estimated to be of the order of 19 mm (3/4 inch). However the transition zone between these two areas is gradual and could be of the order of 1,000 meters or more. The reasons for the wide variation in estimated settlement are as follows: (1) due to highly compressible clay such as Bay mud or fat clay, (2) some clays are overconsolidated by a ratio of 2 or 3 or greater, and (3) thickness of clay layers is highly variable.

6.1 METHOD OF ANALYSIS

The flood levels used in the analysis of each of the selected sections were provided by the District in your letter of January 14, 1999. Reportedly, they are based on 100-year flood levels estimated with HEC-2 simulation of open channel flows. The flood levels are summarized as follows:

Section Number	Station (meters)	100-year Flood Elevation (meters)
1	7 + 500	5.16
2	8 + 610	6.41
3	9 + 570	7.06
4	9 + 600	7.08
5	12 + 210	9.08
6	13 + 500	9.46
7	16 + 620	14.37

The flood levels provided are based on NAVD (metric) datum; to be consistent with our analysis, we converted these values to the NGVD datum.

Seepage through the levee during maximum flood was analyzed using a transient (time-dependent) analysis. A finite element program, SEEP/W [developed by Geo-Slope International (1992)], was used to perform calculations. SEEP/W is a computer program of two-dimensional code with isoparametric and higher order finite elements capable of solving saturated and unsaturated flow conditions using steady state and transient analysis. The program can represent multiple soil types with anisotropic hydraulic conductivity; it can also represent prescribed constant and transient boundary conditions.

As discussed previously in Section 4.2.1 “Design Hydrograph”, the flood levels used in the transient analyses were based on the 1-percent chance flood hydrograph (see Figure 4) for the Guadalupe River developed by the Army Corps of Engineers.

6.2 CROSS SECTIONS AND RESULTS OF ANALYSIS

In general the levees are constructed of well compacted lean clays or lean to fat clays. Furthermore, most of the native soils underlying the levee fills are lean to fat clays with the exception of the East Levee at Boring EB-11. (This location will be discussed further at the end of this section). Since the clayey soils are relatively impervious, it is our engineering opinion that generally (1) seepage through and underneath the levees during the design flood will be relatively small and (2) the potential for piping at the outboard toe will also be very low. To confirm these opinions, we performed a seepage analysis on four sections that represent the following site and subsurface conditions:

- Steep levee side slopes and deep ditch on the outboard toe of levee (Section 2);
- Steep levee side slopes with clayey sand foundation stratum overlain by fat clays (Section 3);
- Silty sand stratum at the bottom of levee fill (Section 4);
- Sand stratum overlain by lean clay foundation of lean clay levee fill (Section 5).

These four sections correspond to the sections listed and described in Section 4.1 "Cross Sections Selected for Analysis".

The coefficients of hydraulic conductivity (permeability) for three types of soil (lean clay, silty sand and sand) were selected from the following laboratory and published data. They included laboratory permeability test results of relatively undisturbed and remolded samples, broad empirical relationships between soil types and permeability (Cedergren, 1967) as shown in Figure 36, and correlation relationships based on grain size distribution as described in the EM-1110-2-1913 (COE, 1978) and the NAVAC DM-7.1 (1982). Permeability tests were performed on three relatively undisturbed soil samples as reported in Appendix G. The coefficient of hydraulic conductivity (permeability), k, determined from these samples are as follows:

Sample Number	**Depth (feet)**	**Soil Type**	**Location**	**Dry Density (pcf)**	**K (cm/second)**
EB 3-16-4	30 – 31.5	Lean Clay	Foundation	101.5	3.3×10^{-7}
EB 11-9-4	14.5 – 16	Silty Sand	Foundation	90.3	5.1×10^{-4}
WB 10-11-4	26 – 27.5	Silty Sand	Foundation	125.2	1.2×10^{-4}

The values assigned to the corresponding soil types are presented in the following table. The ratio of anisotropy of the horizontal to vertical conductivity was assumed to be 4 for all soil types.

Soil Type	**Coefficient of Hydraulic Conductivity (K_h)**		**Selected Values**	
	cm/sec	ft / hr	cm/sec	ft / hr
Sand	10^{-2} to 10^{-4}	1.2 to 0.012	10^{-2}	1.2
Silty Sand, Clayey Sand	5×10^{-3} to 5×10^{-4}	0.7 to 0.07	6×10^{-3}	0.7
Lean to Fat Clay	10^{-5} to 10^{-6}	1.2×10^{-3} to 1.2×10^{-4}	10^{-5}	1.2×10^{-3}

The four sections described above are shown on Figure 37 (Section 2), Figure 38 (Section 3), Figure 39 (Section 4) and Figure 40 (Section 5). These figures also present the estimated flow quantities for a steady state seepage condition with river levels at 100-year elevations, as summarized below:

Figure Number	Section Number	Estimated Seepage Quantity		Comment
		Cubic Feet per Hour per Linear Foot	Gallons per Minute per Linear Foot	
37	2	1.087 x 10^{-2}	1.36 x 10^{-3}	----
38	3	4.754 x 10^{-3}	5.94 x 10^{-4}	----
39	4	7.1806 x 10^{-1}	8.98 x 10^{-2}	Through levee & Foundation
39	4	1.772 x 10^{-3}	2.22 x 10^{-4}	Through levee only
40	5	3.905 x 10^{-3}	4.88 x 10^{-4}	----

For a 30.5-meter (100-foot) long section, the total flow quantity through the levee at each of the above four sections is expected to be less than ¼ gallon per minute (gpm). In our opinion these seepage quantities are very small.

The above discussion is for a steady state condition such that seepage through the levee achieves equilibrium during the 100-year flood elevations. To simulate the change in river level during the flood event, we performed a transient analysis based on the 100-year flood hydrograph (see Figure 4). The results of the transient analysis are presented in Figure 41 (Section 2), Figure 42 (Section 3), Figure 43 (Section 4) and Figure 44 (Section 5). These figures present the phreatic surface through the levee at different times. For example, in Figure 41, the initial groundwater level is shown as line 0; subsequent phreatic surfaces at times of 12 hours (time step 6), 24 hours (time step 12), 48 hours (time step 24), 72 hours (time step 36) and 84 hours (time step 42). As shown on Figure 41, the highest phreatic surface is at 48 hours; this surface is lower than the steady state phreatic surface shown in Figure 37. This observation is similar to the other three sections. By comparison, we conclude that the estimated seepage quantities will be much less than the ¼ gpm estimated previously.

In Figures 41 through 44, most sections indicate thick layers of lean clay soils where seepage exits at the outboard toe of the levee. Due to the cohesiveness of these soils, piping is unlikely.

On the East Levee at Boring EB-11, layers of silty sand and clayey sand about 4.6 meters (15 feet) thick were encountered directly below the levee lean clay fill. Groundwater was encountered within the sand layers at a depth of about 5.8 meters (19 feet) or about 1.5 meters (5 feet) below bottom of fill. For this condition, we have performed a seepage analysis and the flow net is presented in Figure 45. A k value of 5 x 10^{-4} cm/sec was selected for the foundation sand layers, based on the laboratory test on sample EB 11-9-4. In this analysis we assumed the levee clay fill is impervious. Based on this analysis, the seepage quantity is estimated to be 0.16 gpm per linear meter (0.05 gpm per linear foot); for a 30.5 meter (100 feet) long section the total seepage is estimated to be 5 gpm. The estimated exit gradient at the outboard toe is 0.24 and the critical gradient is about 1.0; the latter (EM 1110-2-1901, September 30, 1986, Section 4-9b. "Escape and Critical Gradients") is defined as the ratio of (specific gravity of soil –1) divided by

(1 + void ratio). Therefore, the factor of safety against piping is estimated to be 4. A similar method of calculating the factor of safety against piping was used, as described on pages 226 and 227 in Cedergren (1966). He states the factor of safety, G_s is given by the equation:

$$G_s = \frac{(G-1)/(1+e)}{i}$$

Where G= specific gravity of soil
e= void ratio
i= uplift head divided by resisting soil column weight

This equation also results in a safety factor of about 4. Furthermore, Cedergren also reports observations by Turnbull and Mansur (1961) where they describe underseepage under Mississippi River levees and conclude that "...it appears that heavy underseepage and sand boils should be anticipated whenever estimated upward gradients exceed 0.5 to 0.8, depending on site conditions." Furthermore, they report that if upward gradients are in the range of 0 to 0.5, light to none seepage conditions are anticipated. On that basis for the section shown in Figure 45 we conclude that seepage conditions of light to none are anticipated. Nevertheless, a flow rate of 5 gpm is considered significant and may cause water damage to the outboard levee properties. To effectively reduce the seepage, we need to better define the limits of the sand layer under the East Levee. Therefore, we recommend that additional field exploration be performed, including borings along the crest and at the toe of the levee. Details of the additional field exploration program should be determined during final design.

Soil liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary loss of shear strength due to the cyclic loading associated with earthquake ground shaking. Along creeks and rivers, the principal modes of ground failure generally associated with liquefaction are slumping of stream banks, lateral spreading of flatlands toward open channels and differential ground settlement. Historically, the most common and most damaging ground failures have been lateral spreads (Youd and Hoose, 1978). These failures have been accompanied by considerable ground cracking, differential settlement and eruption of sand boils. The sand boils are evidence that liquefaction played a major role in these ground failures.

The potential for liquefaction at this site was evaluated using semi-empirical methods for a moment magnitude 8.0 earthquake on the San Andreas fault. The basis of this earthquake magnitude and site acceleration is described in the Section 2, "Site Geology and Seismicity."

The clay layers at the site are judged not to be susceptible to liquefaction. Therefore, the potential for liquefaction of various sand and gravel layers at the site was evaluated. On the basis of our analysis, it appears that there is some potential for liquefaction to occur at the locations shown on Figure 46, Location of Potentially Liquefiable Layers. The potentially liquefiable layers appear to be concentrated in the region of the levee alignment north of Gold Street between Station 6+100 to Station 7+100, although scattered liquefiable layers exist along the remainder of the alignment. These layers range in thickness from 0.3 to 4.0 meters (1 to 13 feet). Should liquefaction occur, temporary loss of bearing capacity and settlement could occur at these locations. Settlement of the ground surface is estimated to be on the order of 6 to 76 mm (1/4 to 3 inches) at these locations. Estimates of these potential settlements were made using the procedures outlined by Tokimatsu and Seed, (1984).

Based on explorations conducted, most layers of potentially liquefiable soil were encountered below depths of at least 3 meters (10 feet) below ground surface at landside toe. These deposits are overlain by layers of non-liquefiable cohesive soils. Empirical research by Ishihara, (1985) indicates that for level ground, liquefaction is not likely to result in visible damage at the ground surface as long as the cover of the liquefiable soil is equal to at least 3 meters (10 feet). At the locations of Borings GRIM-1 and RREMB-1 where liquefaction is anticipated, the thickness of non-liquefiable surficial cohesive soils is 2.1 meters (7 feet) and 1.5 meters (5 feet), respectively. Based on Ishihara's research, we expect ground surface rupture even at these two locations is unlikely. In summary, we anticipate the ground surface at or adjacent to the levees will not rupture as a result of liquefaction.

To evaluate the potential for lateral spreading, we reviewed the cross-section geometry and subsurface conditions at each of the locations shown on Figure 46 where liquefaction potential is high. Based on this review, we selected two of the most critical liquefaction sections located as follows:

- Station 7+050 on the East Levee; and
- Station 13+500 on the West Levee.

We performed a wedge slope stability analysis at each of these two sections using the Janbu method. In this analysis, we assumed (1) the horizontal portion of the slide plane is in the top of the liquefied layer, (2) the water level was at a level similar to that encountered during our 1998 field exploration (which is similar to the level in the lower flow channel), (3) a pseudo-static earthquake horizontal force (0.33g) in both the levee and foundation, (4) residual shear strength of the liquefied sand layer consistent with published values (Seed and Harder, 1990), and (5) the residual shear strength occurs simultaneously with the earthquake shaking. The last assumption is somewhat conservative since the residual shear strength condition occurs immediately after the earthquake, i.e. is less than the shear strength during the earthquake. For example, we selected residual shear strength of 14.4 kPa (300 psf) for our analysis; this is less than the static strength of the top of the non-liquefied sand layer. Based on our wedge analysis, the factor of safety during seismic shaking is at least 1.5 at both sections. Therefore, we conclude that ground failure in the lateral spreading mode is unlikely at the liquefaction prone locations shown in Figure 46.

8.1 GENERAL

Based on our site reconnaissance, as well as our subsurface investigation and geotechnical analyses, we believe the geotechnical condition of the existing East and West Levees described herein is generally very good. However, in our opinion there are numerous locations which will require some remediation. In addition there are two sections on the West Levee where slopes should be flattened to increase the factor of safety to minimum design values. These opinions and recommendations are discussed in more detail in this section. It should be understood that our comments are limited to the condition of the existing levees and do not apply to levee segments that might be raised in the future.

8.2 FIELD RECONNAISSANCE

In Section 3.1.1 "Field Reconnaissance", we described the surface conditions of both the East and West Levees observed during our site reconnaissance of December 19, 22 and 23, 1997. Our observations were documented in detail and are presented in Appendix D. Our observations, and the levee repairs that we anticipate could be required, are presented in Tables 9 and 10 for the East Levee and West Levee, respectively. As shown in Tables 9 and 10, the geotechnical conditions have been tabulated for Reaches northwest of G through A into the following six categories:

- Burrow holes
- Seepage
- Slip/Crack
- Settlement
- Erosion
- Others

CH2M Hill has provided the following definitions of each reach used in Tables 9 and 10.

Reach	Station Limits		Description	
	From	To	From	To
G	7+000	8+000	Downstream side of SPRR bridge	Downstream side of Highway 237
F	8+000	9+200	Downstream side of Highway 237	Downstream side of Tasman Drive
E	9+200	10+100	Downstream side of Tasman Drive	Downstream side of Hetch Hetchy
D	10+100	11+800	Downstream side of Hetch Hetchy	Downstream side of Montague Expressway
C	11+800	13+650	Downstream side of Montague Expressway	Downstream side of Trimble Road
B	13+650	14+500	Downstream side of Trimble Road	Downstream side of Highway 101
A	14+500	15+510	Downstream side of Highway 101	Station 15+510

In addition for the East Levee in Table 9, our description included a reach northwest of Reach G from Station 6+100 to 7+000.

As shown in Tables 9 and 10, anticipated repairs range considerably including but not limited to: backfilling burrow holes, replacing over-steepended cut slopes with engineered fill, raising grade in sags, etc. The station locations listed in Tables 9 and 10 are approximate and based on visual observations made during the site reconnaissance; no survey was performed to verify the exact locations estimated in these tables.

8.3 RESULTS OF SLOPE STABILITY EVALUATION

In Section 4 "Slope Stability Evaluation", we discussed (1) the cross sections selected for analysis, (2) method of analysis and (3) stability results. We identified a total of 23 levee sections as candidates for analysis. As requested by the Water District, we then selected what we judged to be the seven most critical sections of the 23. Based on our slope stability analyses of these seven sections, we concluded that five of those sections had factors of safety that exceeded the minimum factor of safety required for five (5) Army Corps of Engineers' design conditions. However, two of these sections on the West Levee, including the outboard slope of Section 2 (Station 8+200 to 8+500) and both the inboard slope and outboard slope of Section 3 (Station 9+230 to 10+100) were found to be slightly deficient in the sudden drawdown and/or steady seepage conditions; at these locations the estimated factor of safety was less than the minimum factor of safety required for design. However, the actual factors of safety were only slightly less than the minimum required, as summarized below:

Levee	Section Number	Station		Slope	Design Condition	Factor of Safety	
		From	To			Minimum Required	Calculated
West	2	8+200	8+500	Outboard	Steady seepage at flood stage	1.4	1.3
West	3	9+230	10+100	Outboard	Steady seepage at flood stage	1.4	1.3
West	3	9+230	10+100	Inboard	Sudden Drawdown	1.0	0.9

These locations are also described in Tables 9 and 10 under the row entitled "Slip/Crack" and column entitled "Anticipated Repair". In summary, the outboard slopes in Sections 2 and 3 and the inboard slope in Section 3 have factors of safety less than the minimum required. Based on this information, we recommend that these slopes be remediated by one or more of the following methods: (1) flattening, (2) berm at toe, (3) floodwall, or (4) reinforced earth wall. The method of remediation will be determined during final design when such details as elevation of raised levee crest and property line location are made available.

Consistent with our contract, we have analyzed seven sections for slope stability, although a total of 23 levee sections were identified. Based on the hydraulic analyses in progress, some of the West and East Levees might be raised. When the final raised heights of both levees are known, additional sections might also need to be analyzed. The need for such analysis should be evaluated as part of the final design studies.

8.4 RESULTS OF SETTLEMENT ANALYSES

In Section 5 "Settlement Evaluations", we discussed (1) land subsidence, (2) inferred levee settlement, (3) predicted settlement and (4) future levee settlement. In evaluating future settlement, we assumed the levee will be raised approximately 0.9 meter (3 feet). On that basis we estimated long term consolidation settlement for raised levees generally north of Montague Expressway to be of the order of 254 mm (10 inches) and south of Montague Expressway to be of the order of 19 mm (3/4 inch). However the transition zone between these two areas is gradual and could be of the order of 1,000 meters or more. The reasons for the wide variation in estimated settlement areas are as follows: (1) due to highly compressible clay such as Bay mud or fat clay, (2) some clays are overconsolidated by a ratio of 2 or 3 or greater, and (3) thickness of clay layers is highly variable.

When crest elevations of raised levees are determined during final design, we recommend that the long term consolidation settlement be reviewed at that time and be revised, if necessary.

8.5 RESULTS OF SEEPAGE ANALYSES

In Section 6 "Seepage Evaluation" we discussed (1) method of analysis and (2) cross sections and results of analysis. In general, the levees are constructed of well compacted lean clays with some localized fat clays. Furthermore, most of the native soils underlying the levee fills are cohesive with the exception of the East Levee at Boring EB-11. Since the cohesive soils are relatively impervious, it is our engineering opinion that generally (1) seepage through and underneath the levees during the design flood will be relatively small and (2) the potential for piping at the outboard toe will also be very low. To confirm these opinions, we performed a seepage analysis on four sections that represent the following site and subsurface conditions:

- Steep levee side slopes and deep ditch on the outboard toe of levee (Section 2);
- Steep levee side slopes with clayey sand foundation stratum overlain by fat clays (Section 3);
- Silty sand stratum at the bottom of levee fill (Section 4);
- Sand stratum overlain by lean clay foundation of lean clay levee fill (Section 5).

For a 30.5-meter (100-foot) long section, the total flow quantity through the levee at each of the above four sections is expected to be less than ¼ gallon per minute (gpm). In our opinion these seepage quantities are very small.

The above discussion is for a steady state condition such that seepage through the levee achieves equilibrium during the 100-year flood elevations. To simulate the change in river level during the flood event, we also performed a transient analysis based on the 100-year flood hydrograph. Based on this analysis, the highest phreatic surface is at 48 hours; this surface is lower than the steady state phreatic surface. This observation is similar to the other three sections. By comparison, we conclude that the estimated seepage quantities probably will be much less than the ¼ gpm estimated.

On the East Levee at Boring EB-11, layers of silty sand and clayey sand about 4.6 meters (15 feet) thick were encountered directly below the levee lean clay fill. Groundwater was encountered within the sand layers at a depth of about 5.8 meters (19 feet) or about 1.5 meters (5 feet) below bottom of fill. For this condition, we performed a seepage analysis. The seepage quantity is estimated to be 0.05 gpm per linear foot; for a 30.5 meter (100 feet) long section the total seepage is estimated to be 5 gpm. The factor of safety against piping is estimated to be 4. A flow rate of 5 gpm is considered significant and may cause water damage to the outboard levee properties. To effectively reduce the seepage, the limits of the sand layer under the East Levee should be better defined. Therefore, we recommend that additional field exploration be performed, including borings along the crest and at the toe of the levee; details of the program should be determined during final design.

8.6 RESULTS OF LIQUEFACTION POTENTIAL

In Section 7 " Liquefaction Potential", we described the results of our liquefaction analysis and identified locations of potentially liquefiable layers in Figure 46. The potentially liquefiable layers appear to be concentrated in the region of the levee alignment north of Gold Street between Station 6+100 to Station 7+100, although scattered liquefiable layers exist along the remainder of the alignment. Should liquefaction occur, temporary loss of bearing capacity and settlement could occur at these locations. Post liquefaction settlement of the ground surface is estimated to be on the order of 6 to 76 mm (1/4 to 3 inches) at these locations. Levees can be overbuilt to accomodate this settlement.

Based on available data and explorations conducted for this study, most layers of potentially liquefiable soil were encountered below depths of at least 3 meters (10 feet) below ground surface at landside toe. These deposits are overlain by layers of non-liquefiable cohesive soils. In summary, we anticipate the ground surface at or adjacent to the levees will not rupture as a result of liquefaction.

To evaluate the potential for lateral spreading, we reviewed the cross-section geometry and subsurface conditions at each of the locations shown on Figure 46 where liquefaction potential appears high. Based on this review, we selected two of the most critical liquefaction sections located as follows:

- Station 7+050 on the East Levee; and
- Station 13+500 on the West Levee.

We performed a slope stability wedge analysis at each of these two sections using the Janbu method. Based on our wedge analysis, the factor of safety during seismic shaking is at least 1.5 at both sections. Therefore, we conclude that ground failure in the lateral spreading mode is unlikely at the liquefaction prone locations shown in Figure 46.

The conclusions and recommendations contained in this report are based on exploratory borings drilled for this study, as well as CPTs, our review of available subsurface information, our site reconnaissance, and our local experience and engineering judgment. The elevations shown on the logs of new borings and CPTs were surveyed and furnished by the District. Furthermore, the slope stability analysis has been limited to the seven (7) critical sections selected. When final flood control improvement alternatives have been selected, more detailed geotechnical evaluations will be necessary to provide the required design parameters. Such evaluations also could require the drilling of new borings, sampling and geotechnical testing.

An investigation for subsurface environmental contamination was beyond the scope of our services.

The recommendations presented in this report were developed with the standard of care commonly used in this profession. No other warranties are included, either expressed or implied, as to the professional advice presented in this report.

Abrahamson, N.A. and Silva, W.J., 1997, Empirical response spectral attenuation relations for shallow crustal earthquakes, Seismological Research Letters, v. 68, p. 94-127.

Boore, D.M., Joyner, W.B., and Fumal, T.E., 1993, Estimation of response spectra and peak accelerations from western North American earthquakes: An interim report, U.S. Geological Survey Open-File Report 93-509, 72 p.

Bureau of Reclamation, U.S. Department of the Interior, 1974, Earth Manual, Second Edition.

Campbell, K.W., 1997, Empirical near-source attenuation relationships for horizontal and vertical components of peak ground acceleration, peak ground velocity, and pseudo-absolute acceleration response spectra, Seismological Research Letters, v. 68, p. 154-179.

Cedergren H.R., 1967, Seepage, Drainage and Flow Notes, 489 p.

Department of the Army Corps of Engineers, 1970, Engineering and Design - Stability of Earth and Rock-Fill Dams, Manual EM-1110-2-1902.

Department of the Army Corps of Engineers, 1978, Design and Construction of Levees, Manual EM-1110-2-1913.

Department of the Army Corps of Engineers, September 30, 1986, Seepage Analysis and Control of Dams, Manual EM-1110-2-1901.

Duncan, J.M. and Buchignani, A.L., 1975, An engineering manual for slope stability studies, University of California, Berkeley, Department of Civil Engineering.

Hitchcock, C.S., Kelson, K.I. and Thompson, S.C., 1994, Geomorphic investigations of deformation along the northeastern margin of the Santa Cruz Mountains, William Lettis & Associates, Inc., Final Technical Report, U.S. Geological Survey National Earthquake Hazards Reduction Program Award No. 1434-92-G-2220, 33 p.

Irwin, W.P., 1990, Geology and plate tectonic development, in The San Andreas Fault System, California, Wallace, R.E. (ed.) U.S. Geological Survey Professional Paper 1515, p. 61-80.

Jennings, C.W., 1994, Fault activity map of California and adjacent areas, California Division of Mines and Geology, California Geologic Data Map Series, Map No. 6, 1:750,000 scale.

Leonards, G.A., 1962, Foundation Engineering, McGraw-Hill Book Company.

Lienkaemper, J.J., Borchardt, G. and Lisowski, M., 1991, Historic creep rate and potential for seismic slip along the Hayward fault, California, Journal of Geophysical Research, v. 96B, p. 18261-18283.

Lienkaemper, J.J. and Galehouse, J.S., 1997, Revised long-term creep rates on the Hayward fault, Alameda and Contra Costa Counties, California, U.S. Geological Survey, Open-File Report 97-690, 18 p.

Poland, J.F., 1971, Land Subsidence in the Santa Clara Valley, Alameda, San Mateo, and Santa Clara Counties, California, U. S. Geological Survey, Map MF - 336.

Rogers, T .H and Williams, J.W., 1974, Potential Seismic Hazards in Santa Clara County, California, California Division of Mines and Geology, Special Report 107, 39p.

Sadigh, K., Chang, C.Y., Egan, J.A., Makdisi, F. and Youngs, R.R., 1997, Attenuation relationships for shallow crustal earthquakes based on California strong motion data, Seismological Research Letters, v. 68, p. 180-189.

Seed, R.B. and Harder, L.F., May, 1990, SPT-Based analysis of cyclic pore pressure generation and undrained residual strength, Volume 2, H. Bolton Seed Memorial Symposium Proceedings.

Tokimatsu, K. and Seed, H.B., 1984, Simplified procedures for the evaluation of settlements in clean sands due to earthquake shaking, Report No. UCB/EERC-84/16, College of Engineering, University of California, Berkeley.

Turnbull, W.J. and Mansur, C.I., 1961, Investigation of Underseepage- Mississippi River Levees, transactions, A.S.C.E., Volume 126, Part 1, pp 1429-1539.

Wallace, R.E.(ed.), 1990, The San Andreas Fault System, California, USGS, Professional Paper 1515, 283 p.

Working Group on California Earthquake Probabilities, 1990, Probabilities of large earthquakes in the San Francisco Bay region, California, U.S. Geological Survey, Circular 1053, 51 p.

Working Group on Northern California Earthquake Potential, 1996, Database of potential sources for earthquakes larger than magnitude 6 in northern California, U.S. Geological Survey, Open-File Report 96-705, 53 p.

Wright, S.G., 1991, UTEXAS3 – A computer program for slope stability calculations, Shinoak Software, Austin, Texas.

Youd, T.L. and Hoose, S.N., 1978, Historic Ground Failures in Northern California Triggered by Earthquakes; U.S. Geological Survey Professional Paper 993.

TABLE 1
Existing Boring Log Index List
Lower Guadalupe River Flood Control Project
Santa Clara County, California

Boring Index	Consultants	Consultants' Project Number	Project Date	Project Name
AA-n	Woodward-Clyde	973061NA	Sept 1997	Preliminary Investigation, Fuel Storage and Dispensing Facility for American Airlines, San Jose
BSP-n	Woodward-Clyde	4327	Dec 1960	Soil Investigation for the Proposed Bay Shell Plant Storage Tanks, San Jose
GJJ-n	Gribaldo,Jacobs, Jones & Associates	E584-M1	Jan 1963	Soil Investigation for Guadalupe River Project, Reach No. 1, Preliminary Study of the West Bank of Alviso Slough and Proposed SPRR Retaining Wall, Santa Clara County
GRI-n	John V. Lowney & Associates	109-11 PA 9669	Jan 1981	Geotechnical Investigation, Guadalupe River Improvement, San Jose
GRIM-n	Woodward-Clyde	15482V	Oct 1982	Geotechnical Investigation, Guadalupe River Flood Control Improvements, Southern Pacific Railroad to the County Marina in Alviso
GRIV-n	Woodward-Clyde	4246	c. 1960	Soil Investigation for the Proposed Guadalupe River Project, Reaches 1 and 2, Santa Clara County
GROCS-n	John V. Lowney & Associates	109-12 PA 11516	June 1983	Geotechnical Investigation, Guadalupe River Outfall Control Structure, San Jose
GSB-n	Woodward-Clyde	16338V	Oct 1984	Geotechnical Investigation, Raising of Gold Street Bridge over the Guadalupe River in Alviso, San Jose
GSBOLD-n	Woodward-Clyde	4246	c. 1960	Gold Street Bridge Investigation as part of the 1960 Guadalupe River Project
GTPP-n	Parikh Consultants, Inc.	953009NE	c. 1996	Golden Triangle Pipeline Project, South Bay Water Recycling Program
HHP-n	Woodward-Clyde	91C0639R	Jan 1992	"Hetch-Hetchy Pipelines," Relocation of San Francisco Water Department, Bay Division Pipeline Numbers 3 and 4 at Guadalupe River and Coyote Creek, San Jose and Santa Clara
HHPA-n	PSC Associates, Inc.	88122.10	July 1988	Lick Mill Blvd. Extension, Santa Clara
OPS-n	Woodward-Clyde	16784V	c. 1985	Geotechnical Consultation, Oakmead Storm Pump Station Outfall Spillway

TABLE 1
Existing Boring Log Index List
Lower Guadalupe River Flood Control Project
Santa Clara County, California

Boring Index	**Consultants**	**Consultants' Project Number**	**Project Date**	**Project Name**
PRMAR-n	Soil Mechanics & Foundation Engineers	1140	Feb 1962	Soil Engineering Report for the Proposed Marina, Alviso, Santa Clara County
PYH-n	Cooper & Clark	33A	Jan 1960	Preliminary Site Investigation, Proposed Yacht Harbor, Alviso
RBPS-n	Woodward-Clyde	16171X	Feb 1987	Geotechnical Investigation, Retention Basin and Pump Station, San Jose International Airport, San Jose
REAL1-n	Woodward-Clyde	5135	June 1962	Soil Investigation, Guadalupe River Project, Realignment within Reach 1, Santa Clara
RHM-n	Woodward-Clyde	92C0526R	Nov 1992	Soil Investigation, Riparian Habitat Mitigation, Route 87
RREMB-n	John V. Lowney & Associates	109-1 PA 1233	Oct 1969	Soil Mechanics Investigation, SPRR Bridge and Embankment Raising over Guadalupe River, San Jose
SBYC-n	Woodward-Clyde	15482-W-2000	April 1983	Geotechnical Study, South Bay Yacht Club Relocation, Alviso
SDB-n	Woodward-Clyde	963010NA	c. 1997	Geotechnical Investigation, Skyport Drive Bridge, San Jose
SIL-n	Cooper & Clark	170-G	Oct 1982	Soil Investigation, Landslide Areas, Guadalupe River, San Jose
SJIA-n	Woodward-Clyde	16171V	Feb 1984	Geotechnical Investigation, San Jose International Airport, San Jose
SPRR-n	Jo Crosby & Associates	2706	Jan 1976	Soils Investigation for Guadalupe River, SPRR to Trimble Road, Project # 3015
SPRRB-n	Woodward-Clyde	4246	Jan 1961	Soil Investigation for the Proposed Southern Pacific Company Bridge, Alviso (in conjunction with soil investigation of Guadalupe River Project, Reaches 1 and 2)
TDB-n	Woodward-Clyde	16266V	May 1984	Geotechnical Study, Tasman Drive Bridge at Guadalupe River, Santa Clara/San Jose

Note:

1. The value "n" in the boring index column represents the Boring Number used in the corresponding project, such as 1 and B1.

TABLE 2
Notes From Meeting On December 9, 1997
Lower Guadalupe River Flood Control Project
Santa Clara County, California

Location No.	Approximate Channel Station	Approximate Date Observed or Occurrence of Incidents	Description of Problem	Repair or mitigation
1	07+130 to 07+980 West Levee	1986, 1992 and 1996	Minor cracking along west levee from Gold Street Bridge to Hwy 237.	Regraded crest.
2	07+150 East Levee, east of Gold Street Bridge	1986 or 1988	During raising of Gold Street Bridge, asbestos was revealed in an excavation made in the vicinity of an old meander. The asbestos was later capped with about 6 feet of silt near outboard toe. No reported post-construction problems.	
3	08+270 East Levee	1986	Localized sloughing on outboard slope (East Levee) after heavy rain	
4	08+630 to 08+670 East Levee	Mid 1980s	Erosion of Oakmead Pump Station outfall.	Placed large rock riprap at inboard toe of outfall.
5	09+300 to 10+150 West Levee	On-going	Tire ruts left by County trucks and neighborhood recreational motorbikes	
6	09+300 to 11+800 East Levee	On-going	Water seeps through conduits in upper levee apparently created by animal burrows and dead tree (eucalyptus) root networks.	

TABLE 2
Notes From Meeting On December 9, 1997
Lower Guadalupe River Flood Control Project
Santa Clara County, California

Location No.	Approximate Channel Station	Approximate Date Observed or Occurrence of Incidents	Description of Problem	Repair or mitigation
7	09+550 West Levee	On-going	Return flow during high river level: improperly shut slide gate due to vandalism and sediment blocking outfall pipe.	Periodic cleaning of pipes
8	09+840 East Levee	Spring 1995	Blowout at Cisco System parking lot; asphalt pavement was lifted about 1 foot. Cause: Old drain pipe for "flood" irrigation was apparently capped improperly during construction of Cisco System campus. Cap was forced open during high river level in Spring 1995.	Exposed drain pipe at the levee toe on the inboard side and filled with concrete.
9	10+150 to 10+910 West Levee	1994 to 1995	Leaking wells due to artesian water conditions. Wells are either old farmer wells or monitoring wells for former CAMSL III site. Old well at about Station 10+500 ± at outboard toe to be destroyed.	Destroyed wells as they were uncovered.
10	10+900 West Levee	On-going	Return flow during high river level: improperly shut slide gate due to vandalism and sediment blocking outfall pipe.	

TABLE 2
Notes From Meeting On December 9, 1997
Lower Guadalupe River Flood Control Project
Santa Clara County, California

Location No.	Approximate Channel Station	Approximate Date Observed or Occurrence of Incidents	Description of Problem	Repair or mitigation
11	11+090 West Levee	Spring 1995	Slope failure on outboard side of levee adjacent to Lick Mill development: · wet spots were observed; · material became saturated and was described as "viscous;" · 50 to 70-foot section of levee slipped (approx. 3/4 width of levee). Debris including bricks and plastic bottles were exposed in the levee fill. Possible cause: artesian water condition at an old well suspected to be part of the monitoring well network for a former chemical plant, CAMSL III (1983 plan)	Replaced damaged levee with clean, compacted fill. Well Section should have kept records of well and its destruction. Contact: Mike Duffey
12	11+160 to 11+200 West Levee	Fall 1997	Seepage observed on inboard toe of West Levee. Cause: artesian water condition from an old well under levee fill.	Sealed well. Removed wet fill, including brick and concrete chunks, pipes and other debris, and replaced with well compacted import clay fill.

TABLE 2
Notes From Meeting On December 9, 1997
Lower Guadalupe River Flood Control Project
Santa Clara County, California

Location No.	Approximate Channel Station	Approximate Date Observed or Occurrence of Incidents	Description of Problem	Repair or mitigation
13	11+800 East Levee	August 1996	Deep crack on outboard side of levee and shallow slip plane near old outfall pipe (20 inch dia.)	
14	12+000 West Levee	1982 to 1983	50 to 100-foot section on the inboard toe of old levee eroded	Backfilled with rock
15	13+150 East Levee	1995	Blowout at Acer parking lot beyond outboard toe of levee; lifted asphalt concrete pavement about 2 feet. Cause: Improperly plugged old irrigation drain pipe was forced open during high river flow level in Spring 1995.	Exposed drain pipe at the levee toe on the inboard side and filled with concrete.
16	13+650 to 13+750 West Levee	Late 1980s and 1996	Surface cracking observed. Cause: Shrinkage cracks in late 1980s apparently the result of drought conditions.	
17	14+350 East Levee	1980	Erosion along levee bank downstream of concrete lined slope	Scraped off loose material and recompacted slope surface.
18	16+100 to 16+400 West Levee	On-going and more significantly in 1995	Collapse of maintenance road bank primarily due to scouring and erosion.	Addressed in new Skyport Drive Bridge slope protection with proposed gabion wall.

TABLE 2
Notes From Meeting On December 9, 1997
Lower Guadalupe River Flood Control Project
Santa Clara County, California

Other observations:

1. Between 1993 and 1994, the District excavated about 14,000 cubic yards of sediments in low flow area in the reach from Montague Expressway to about 200 feet upstream of Trimble Road.
2. Previous settlement data was based on NGVD datum; current and new data are based on North American Vertical Datum.
3. During 1995 raising of levees between Highway 101 and south of Gold Street, reportedly truck traffic caused pumping/rutting of clay materials on crest at some locations.

Notes:

1. The information presented in the above table was based on a meeting with the Santa Clara Valley Water District (SCVWD) staff on Dec 9, 1997. Those who were present at the meeting included:

SCVWD	Peter Sheydayi
	Darin Gossett
	Dennis Ely
	Rick Linquist
	Gene Balzer
	Kenn Reiller
Woodward-Clyde	Michael Larson
	Stephen Huang

TABLE 3a

ESTIMATED MEDIAN (50th PERCENTILE) PEAK HORIZONTAL ACCELERATIONS IN g's AT SEGMENT 1

Fault	Maximum Magnitude (M_w)	Sense of Faulting[1]	Distances			Peak Ground Acceleration (g)				
			Horizontal Distance[2] (km)	Seismogenic Distance[3] (km)	Rupture Distance[4] (km)	Campbell (1997)	Abrahamson & Silva (1997)	Boore *et al.* (1993)	Sadigh *et al.* (1997)	**Average**
Monte Vista - Shannon	6¾	R	13.8	15.9	13.8	0.30	0.28	0.21	0.31	**0.27**
Hayward - SE Extension	6½	OR	9.4	11.6	9.4	0.32	0.31	0.23	0.32	**0.30**
Southern Hayward	7	SS	10.6	10.8	10.6	0.37	0.29	0.28	0.31	**0.31**
Calaveras	6½	SS	13.8	14.0	13.8	0.24	0.21	0.18	0.22	**0.21**
San Andreas - Peninsula Segment	7	SS	20	20.1	20	0.24	0.18	0.18	0.21	**0.20**
San Andreas - Santa Cruz Mountains Segment	7	SS	20	20.1	20	0.24	0.18	0.18	0.21	**0.20**
San Andreas - 1906 Rupture	8	SS	20	20.1	20	0.39	0.25	0.31	0.30	**0.31**
Sargent	6¾	OR	17.8	19.9	17.8	0.22	0.21	0.17	0.23	**0.21**

[1] R - reverse or thrust fault; OR - oblique-reverse fault; SS - strike-slip fault.

[2] M_w and horizontal distance used by Boore *et al.* (1993). Horizontal distance is defined as the shortest distance from the site to the vertical projection of the fault rupture on the earth's surface.

[3] M_w and seismogenic distance used by Campbell (1997). Seismogenic distance is the shortest distance from the site to the zone of seismogenic rupture. The top of this zone is assumed to be at a depth of 2 km.

[4] M_w and rupture distance used by Abrahamson and Silva (1997) and Sadigh *et al.* (1997). Rupture distance is the shortest distance from the site to the surface rupture.

TABLE 3b

ESTIMATED MEDIAN (50th PERCENTILE) PEAK HORIZONTAL ACCELERATIONS IN g's AT SEGMENT 2

			Distances			Peak Ground Acceleration (g)				
Fault	Maximum Magnitude (M_w)	Sense of Faulting[1]	Horizontal Distance[2] (km)	Seismogenic Distance[3] (km)	Rupture Distance[4] (km)	Campbell (1997)	Abrahamson & Silva (1997)	Boore *et al.* (1993)	Sadigh *et al.* (1997)	**Average**
Monte Vista - Shannon	6¾	R	14.4	16.4	14.4	0.29	0.27	0.20	0.30	**0.27**
Hayward - SE Extension	6½	OR	9.4	11.6	9.4	0.32	0.31	0.23	0.32	**0.30**
Southern Hayward	7	SS	11.9	12.1	11.9	0.35	0.27	0.26	0.29	**0.29**
Calaveras	6½	SS	13.1	13.3	13.1	0.25	0.22	0.19	0.23	**0.22**
San Andreas - Peninsula Segment	7	SS	20	20.1	20	0.24	0.18	0.18	0.21	**0.20**
San Andreas - Santa Cruz Mountains Segment	7	SS	20	20.1	20	0.24	0.18	0.18	0.21	**0.20**
San Andreas - 1906 Rupture	8	SS	20	20.1	20	0.39	0.25	0.31	0.30	**0.31**
Sargent	6¾	OR	17.5	19.6	17.5	0.22	0.21	0.18	0.23	**0.21**

[1] R - reverse or thrust fault; OR - oblique-reverse fault; SS - strike-slip fault.

[2] M_w and horizontal distance used by Boore *et al.* (1993). Horizontal distance is defined as the shortest distance from the site to the vertical projection of the fault rupture on the earth's surface.

[3] M_w and seismogenic distance used by Campbell (1997). Seismogenic distance is the shortest distance from the site to the zone of seismogenic rupture. The top of this zone is assumed to be at a depth of 2 km.

[4] M_w and rupture distance used by Abrahamson and Silva (1997) and Sadigh *et al.* (1997). Rupture distance is the shortest distance from the site to the surface rupture.

TABLE 3c

ESTIMATED MEDIAN (50th PERCENTILE) PEAK HORIZONTAL ACCELERATIONS IN g's AT SEGMENT 3

Fault	Maximum Magnitude (M_W)	Sense of Faulting[1]	Distances			Peak Ground Acceleration (g)				
			Horizontal Distance[2] (km)	Seismogenic Distance[3] (km)	Rupture Distance[4] (km)	Campbell (1997)	Abrahamson & Silva (1997)	Boore *et al.* (1993)	Sadigh *et al.* (1997)	**Average**
Monte Vista - Shannon	6¾	R	13.8	15.9	13.8	0.30	0.28	0.21	0.31	**0.27**
Hayward - SE Extension	6½	OR	9.4	11.6	9.4	0.32	0.31	0.23	0.32	**0.30**
Southern Hayward	7	SS	13.8	13.9	13.8	0.32	0.24	0.24	0.27	**0.27**
Calaveras	6½	SS	13.8	13.9	13.8	0.24	0.21	0.18	0.22	**0.21**
San Andreas - Peninsula Segment	7	SS	19.5	19.6	19.5	0.25	0.19	0.19	0.21	**0.21**
San Andreas - Santa Cruz Mountains Segment	7	SS	19.5	19.6	19.5	0.25	0.19	0.19	0.21	**0.21**
San Andreas - 1906 Rupture	8	SS	19.5	19.6	19.5	0.40	0.25	0.31	0.31	**0.32**
Sargent	6¾	OR	16.9	19.0	16.9	0.23	0.22	0.18	0.24	**0.22**

[1] R - reverse or thrust fault; OR - oblique-reverse fault; SS - strike-slip fault.

[2] M_w and horizontal distance used by Boore *et al.* (1993). Horizontal distance is defined as the shortest distance from the site to the vertical projection of the fault rupture on the earth's surface.

[3] M_w and seismogenic distance used by Campbell (1997). Seismogenic distance is the shortest distance from the site to the zone of seismogenic rupture. The top of this zone is assumed to be at a depth of 2 km.

[4] M_w and rupture distance used by Abrahamson and Silva (1997) and Sadigh *et al.* (1997). Rupture distance is the shortest distance from the site to the surface rupture.

TABLE 3d

ESTIMATED MEDIAN (50th PERCENTILE) PEAK HORIZONTAL ACCELERATIONS IN g's AT SEGMENT 4

Fault	Maximum Magnitude (M_w)	Sense of Faulting[1]	Distances			Peak Ground Acceleration (g)				
			Horizontal Distance[2] (km)	Seismogenic Distance[3] (km)	Rupture Distance[4] (km)	Campbell (1997)	Abrahamson & Silva (1997)	Boore *et al.* (1993)	Sadigh *et al.* (1997)	**Average**
Monte Vista - Shannon	6¾	R	12.5	14.6	12.5	0.33	0.30	0.22	0.33	**0.30**
Hayward - SE Extension	6½	OR	9.4	11.6	9.4	0.32	0.31	0.23	0.32	**0.30**
Southern Hayward	7	SS	16.3	16.4	16.3	0.29	0.22	0.21	0.24	**0.24**
Calaveras	6½	SS	13.1	13.3	13.1	0.25	0.22	0.19	0.23	**0.22**
San Andreas - Peninsula Segment	7	SS	18.8	18.9	18.8	0.25	0.19	0.19	0.22	**0.21**
San Andreas - Santa Cruz Mountains Segment	7	SS	18.8	18.9	18.8	0.25	0.19	0.19	0.22	**0.21**
San Andreas - 1906 Rupture	8	SS	18.8	18.9	18.8	0.41	0.26	0.32	0.31	**0.33**
Sargent	6¾	OR	16.3	18.4	16.3	0.24	0.22	0.18	0.24	**0.22**

[1] R - reverse or thrust fault; OR - oblique-reverse fault; SS - strike-slip fault.

[2] M_w and horizontal distance used by Boore *et al.* (1993). Horizontal distance is defined as the shortest distance from the site to the vertical projection of the fault rupture on the earth's surface.

[3] M_w and seismogenic distance used by Campbell (1997). Seismogenic distance is the shortest distance from the site to the zone of seismogenic rupture. The top of this zone is assumed to be at a depth of 2 km.

[4] M_w and rupture distance used by Abrahamson and Silva (1997) and Sadigh *et al.* (1997). Rupture distance is the shortest distance from the site to the surface rupture.

TABLE 4
SUBSURFACE CHARACTERIZATION
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT

Levee	Levee Section Number	Station Limits		Comments
		From	To	
West	W1	7+000	7+450	
	W2	7+450	8+200	Common with E2
	W3	8+200	8+500	
	W4	8+500	9+230	
	W5	9+230	10+100	
	W6	10+100	11+730	
	W7	11+730	13+000	
	W8	13+000	14+040	
	W9	14+040	14+570	Common with E9
	W10	14+570	15+450	Common with E10
	W11	15+450	15+900	
	W12	15+900	16+770	
	W13	16+770	17+550	
East	E1	6+200	6+640	
	E2	6+640	8+050	Common with W2
	E3	8+050	9+550	
	E4	9+550	10+140	
	E5	10+140	11+100	
	E6	11+100	11+440	
	E7	11+440	12+450	
	E8	12+450	13+650	
	E9	13+650	14+850	Common with W9
	E10	14+850	15+56	Common with W10

TABLE 5
PROPOSED 7 CROSS SECTIONS OF LEVEES CONSIDERED TO BE MOST CRITICAL (11/10/98)

Cross Section Number	Relevant Levee Section Number	Section Location	Critical Boring(s)	Comments
1	E2 W2	East Levee	EB-02 WB-02	Foundation soft Bay Mud found in EB-02 and WB-02 has very low shear strength
2	W3	West Levee	WB-03 WB-04	Ditch at toe of outboard slope
3	W5	West Levee	WB-05	Soft Fat CLAY (CH)
4	E4	East Levee	GRIV-8	Very loose Silty SAND (SM)
5	W7	West Levee	WB-08 WB-09	Steep side slopes observed in field
6	W8	West Levee	WB-13	Very loose Silty SAND (SM)
7	W12	West Levee	WB-16	Medium to Fat CLAY (CH) and steep side slopes

TABLE 6
SUMMARY OF EFFECTIVE SHEAR STRENGTH PARAMETERS BASED ON ISOTROPICALLY CONSOLIDATED UNDRAINED TRIAXIAL TESTS

SOIL TYPE	DRY DENSITY RANGE (pcf)	ESTIMATED EFFECTIVE STRENGTH PARAMETERS							
		ϕ' (degrees)				c' (psf)			
		Peak	10% ε	20% ε	Design	Peak	10% ε	20% ε	Design
Lean CLAY FILL	Less than 110	23.7	22	16.3	24	456	466	574	450
Lean CLAY FILL	Greater than 110	32.2	33.8	30.1	33	0	0	0	0
Fat CLAY	Less than 80	19.8	12.2	NA	20	113	214	NA	110
Lean CLAY	From 80 to 100	27.8	26.3	22.8	28	100	253	251	100
Lean CLAY	From 100 to 110	29.7	29.8	21.8	30	175	175	824	175

Note: Definitions for attached figures are

$q = \frac{1}{2} (\sigma_1' - \sigma_3')$

$p = \frac{1}{2} (\sigma_1' + \sigma_3')$

TABLE 7

ARMY CORPS. OF ENGINEERS
SLOPE STABILITY DESIGN REQUIREMENTS

Case Number	Design Condition	Slope Analyzed	Shear Strength	Minimum Factor of Safety
I	End of Construction	Riverside and Landside	Q or S	1.3
II	Sudden Drawdown	Riverside	S where < R R where < S	1.0
III	Critical Flood Stage	Riverside	S where < R R+S/2 where R<S	1.4
IV	Steady Seepage at Flood Stage	Landside	S where < R R+S/2 where R<S	1.4
VI	Earthquake	Riverside and Landside		1.0

TABLE 8

SUMMARY OF SLOPE STABILITY FACTORS OF SAFETY

Design Condition	**Min. F.S.**	**Section 1**	**Section 2**	**Section 3**	**Section 4**	**Section 5**	**Section 6**	**Section 7**
End of Construction	1.3	3.6	3.3	3.9	4.2	4.0	6.8	10.2
Sudden Drawdown	1.0	1.1	1.0	0.9	2.6	2.5	1.0	2.7
Critical Flood Stage	1.4	1.5	1.8	3.4	4.0 h	6.1 h	2.4	3.4 h
Steady Seepage at Flood Stage	1.4	1.6	1.3	1.3	2.3	5.8	1.7	11.5
Earthquake	1.0	1.2	1.2	1.3	1.2	1.3	2.1	4.4

TABLE 9
SITE RECONNAISSANCE SUMMARY OF EAST LEVEE
December 19, 22, and 23, 1997
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT
95-973017NB.00

Station	Observed Condition Category	Location on Surface of Levee [1,2]			
		Inboard Slope	Crest	Outboard Slope	Anticipated Repair
Northwest of Reach G					
6+100 to 7+000	Burrow Holes	Near Sta. 6+310, 6+450, 6+540, 6+640, and 6+690.	Near Sta. 6+280.	Clusters at Sta. 6+390 and 6+650. At 6+450 and 6+540.	Backfill burrow holes.
	Seepage	Wet spot at toe at Sta. 6+450.			None
	Slip/Crack	Disposed mud on slope at Sta. 6+180. Steepened slope at toe (1:1) between Sta. 6+330 and 6+360.			Flatten slope at toe with engineered fill between Sta. 6+330 to 6+360 to match adjacent slope.
	Settlement		Tire rut at Sta. 6+180. Sagged on outboard edge at Sta. 6+540.		On half width of crest excavate top 460 mm and replace (tire rut) with engineered fill between Sta. 6+170 to 6+190.
	Erosion				None
	Others	30m x 5m old concrete berth at Sta. 6+270 unsorted debris.			None
Reach G (From the downstream side of SPRR bridge to downstream side of Highway 237)					
7+000 to 8+000	Burrow Holes	Near Sta. 7+040, 7+090 (150mm dia.), 7+150, 7+250, 7+410, 7+460, and 7+870.		Near Sta. 7+090 (150mm dia), 7+660, and 7+720.	Backfill burrow holes.
	Seepage	Wet spot at toe at Sta. 7+550 and between Sta. 7+690 and 7+710.			None
	Slip/Crack		Cracks at Sta. 7+160.		Between Sta. 7+150 to 7+170 excavate top 300 mm of levee fill on crest and replace with engineered fill.
	Settlement		Slightly depressed near edge between Sta. 7+150 and 7+170.		None
	Erosion	Sheet erosion between Sta. 7+690 and 7+710. Rill at toe at Sta. 7+845.			Fill rill with engineered fill.
	Others	Concrete debris near Sta. 7+000. Willow tree near Sta. 7+490.			Remove concrete debris. Remove willow tree and stump and backfill hole with engineered fill.
Reach F (From the downstream side of Highway 237 to downstream side of Tasman Drive)					
8+000 to 9+200	Burrow Holes	Cluster between Sta. 8+160 and 8+200. Scattered between Sta. 8+500 and 9+200.		Cluster between Sta. 8+160 and 8+200. Near Sta. 8+240. Scattered between Sta. 8+500 and 9+200	Backfill burrow holes.
	Seepage			Wet spots at toe near Sta. 8+210, and between Sta. 8+250 and 8+400.	None

Note 1. Station locations listed are approximate values based on visual estimates.
Note 2. Conditions were observed on December 19, 22 and 23, 1997.

TABLE 9 (continued)
SITE RECONNAISSANCE SUMMARY OF EAST LEVEE
December 19, 22, and 23, 1997
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT
95-973017NB.00

Station	Observed Condition Category	Location on Surface of Levee [1,2]			
		Inboard Slope	Crest	Outboard Slope	Anticipated Repair
8+000 to 9+200 (continued)	Slip/Crack			½ m wide and ½ m deep gouge at toe near Sta. 8+100. Stockpiled material excavated from ½m cut at toe and nearby condominium development between Sta. 8+700 and 9+150.	Fill gouge and toe cut with engineered fill. Excavate toe at least 1 meter high and 1 meter into slope at toe of slope between Sta. 8+700 to 9+150 and immediately backfill with engineered fill.
	Settlement	Pothole near Sta. 8+430.			Excavate pothole to depth of 450 mm at Sta. 8+430 and replace with engineered fill.
	Erosion			Torn erosion control mat at Sta. 8+100 and 8+575.	Replace with new erosion control mat and engineered fill.
	Others				
Reach E (From the downstream side of Tasman Drive to downstream side of Hetch Hetchy)					
9+200 to 10+100	Burrow Holes	Near Sta. 9+310, 9+480, 9+500, and 9+770. Cluster between Sta. 9+530 and 9+570 and near Sta. 10+040. Trail at toe near Sta. 10+100.	Near Sta. 9+550.	Cluster near Sta. 9+340 and between Sta. 10+030 and 10+050. Near Sta. 9+470 and 9+780. Undermined flood wall near Sta. 9+530, 9+650, and 9+880.	Backfill burrow holes.
	Seepage			Wet spot at toe between Sta. 9+670 and 9+680.	None
	Slip/Crack	½m cut at toe between Sta. 9+680 and 9+720. Shallow slip at toe between Sta. 9+970 and 9+990.		Excavation for pipe installation between Sta. 9+200 and 9+230; undermined slope toe more than 1m.	Both inboard and outboard slopes: Fill toe cuts with engineered fill and match adjacent grades. Inboard Slope: Remove shallow slip at toe of inboard slope between Sta. 9+970 to 9+990 and replace with engineered fill; excavate trapezoid equal to equipment width from midpoint of slope to toe.
	Settlement			Soft spot at toe near Sta. 9+430. Soft with high moisture at toe between Sta. 9+480 and 9+540.	None
	Erosion	Rill near crest between Sta. 9+530 and 9+570. Gully at toe near Sta. 9+650. Gully below burrow holes near Sta. 9+770.	Path created by foot traffic has disturbed levee fill near the beginning of concrete flood wall at Sta. 9+575.	Damaged erosion control mat between Sta. 9+400 and 9+550. Rill below burrow holes near Sta. 9+780.	Replace with new erosion control mat. Fill rills and gullies with engineered fill.
	Others				

Note 1. Station locations listed are approximate values based on visual estimates.
Note 2. Conditions were observed on December 19, 22 and 23, 1997.

TABLE 9 (continued)
SITE RECONNAISSANCE SUMMARY OF EAST LEVEE
December 19, 22, and 23, 1997
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT
95-973017NB.00

Station	Observed Condition Category	Location on Surface of Levee [1, 2]			
		Inboard Slope	Crest	Outboard Slope	Anticipated Repair
Reach D (From the downstream side of Hetch Hetchy to downstream side of Montague Expressway)					
10+100 to 11+800	Burrow Holes	Cluster near Sta. 10+125, 10+260 and 10+305. Near Sta. 10+380 (large hole), 10+425, 10+620 and 10+830. Cluster near Sta. 11+225 and 11+325. Near Sta. 11+580 and 11+750.	Near Sta. 10+850.	Cluster and holes near Sta. 10+440 and 10+500, respectively. Cluster between Sta. 10+700 and 10+900. Near Sta. 10+920 (loose soil pile below holes). Near Sta. 11+420, 11+435, 11+450, 11+680 and 11+720.	Backfill burrow holes.
	Seepage	Well at toe near Sta. 11+050.			District to determine if well at Sta. 11+050 is still active; if inactive District should abandon it in accordance with District standards.
	Slip/Crack	½m cut at toe between Sta. 10+260 and 10+270. Steepened toe (old slip) near Sta. 10+650. Shallow slip near Sta. 10+680. ¾m cut at toe from Sta. 11+180 to 11+200. Shallow slipout between Sta. 11+440 and 11+460.	Slight separation at flood wall and fill interface from Sta. 10+120 to 10+130 and from Sta. 10+190 to 10+200.	150mm undercut of toe between Sta. 11+580 and 11+590. Longitudinal cracks near crest from Sta. 11+780 to 11+800.	Inboard slope: Fill cut at toe between Sta. 10+260 and 10+270 and between Sta. 11+180 to 11+200. Remove slope and replace with engineered fill near Sta. 10+650, 10+680 and between Sta. 11+440 to 11+460; excavate trapezoid to equipment width parallel to slope face from crest to toe. Crest: Fill separation at flood wall and fill interface with engineered fill. Outboard slope: Fill cut at toe between Sta. 11+580 to 11+590 with engineered fill. Where crack is located on slope face, excavate trapezoid equal to equipment width parallel to slope face from crest to 3 meters below hinge point. Between Sta. 11+780 to 11+800, excavate trapezoid to equipment width parallel to slope face from crest to toe. Replace with engineered fill.
	Settlement		Minor sagging between Sta. 11+325 and 11+345.		Raise grade in sag with engineered fill.
	Erosion	Sheet erosion between Sta. 11+150 and 11+180. Gully at toe from Sta. 11+180 to 11+200.	Gully below burrow holes near Sta. 10+850 near edge of levee.	1m strip surface erosion at Sta. 10+270. 100 to 150mm deep gully near Sta. 11+300.	Fill gullies with engineered fill.
	Others	Tree stump at Sta. 11+350.			Remove tree stump and backfill with engineered fill.
Reach C (From the downstream side of Monague Expressway to downstream side of Trimble Road)					
11+800 to 13+650	Burrow Holes	Cluster near Sta. 12+805. Near Sta. 12+845, 12+855, 12+945, and 12+995.		Cluster from Sta. 12+000 to 12+010, and near Sta. 12+200, 12+575 and 13+430. At edge of crest near Sta. 12+710.	Backfill burrow holes.
	Seepage				None

Note 1. Station locations listed are approximate values based on visual estimates.
Note 2. Conditions were observed on December 19, 22 and 23, 1997.

TABLE 9 (continued)
SITE RECONNAISSANCE SUMMARY OF EAST LEVEE
December 19, 22, and 23, 1997
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT
95-973017NB.00

Station	Observed Condition Category	Location on Surface of Levee [1, 2]			
		Inboard Slope	Crest	Outboard Slope	Anticipated Repair
11+800 to 13+650 (continued)	Slip/Crack	Steeper toe slope from Sta. 12+005 to 12+200. ½m cut at toe from Sta. 12+200to 12+220. Shallow slip with 1m cut at toe near Sta. 12+480, 12+510, and 12+540. Shallow slip at toe between Sta. 12+625 and 12+635, and between Sta. 12+660 and 12+700. Shallow slip near Sta. 12+710. Shallow slip at toe from Sta. 13+000 to 13+100, near Sta. 13+110 and 13+160.		3m longitudinal crack near Sta. 13+320.	Inboard slope: Flatten toe of slope with engineered fill between Sta. 12+005 to 12+200 and 12+200 to 12+220; match adjacent grades. At slip locations, excavate trapezoid equal to equipment width parallel to slope face from toe to crest ; replace with engineered fill. Outboard slope: Between Sta. 13+315 to 13+325 at crack, excavate trapezoid equal to equipment width parallel to slope face from toe to crest; replace with engineered fill.
	Settlement				None
	Erosion	Scour at toe near Sta. 13+340.		Rill and gully near Sta. 11+880 formed by disturbance from foot traffic.	Fill scoured toe with engineered fill near Sta. 13+340. Fill rill and gully near Sta. 11+880 with engineered fill.
	Others			Landscaped with trees on slope from Sta. 11+800 to 12+100 and from Sta. 12+400 to 13+650.	Remove trees and stumps and backfill with engineered fill.
Reach B (From the downstream side of Trimble Road to downstream side of Highway 101)					
13+650 to 14+500	Burrow Holes			Cluster near Sta. 14+035.	Backfill burrow holes.
	Seepage	Eight weep holes located on concrete slab between Sta. 14+350 and 14+500; two weep holes are plugged.			Clean out plugged weep holes.
	Slip/Crack	Bulged at toe from Sta. 14+180 to 14+200. Shallow slip from Sta. 14+335 to 14+350.		Shrinkage cracks from Sta. 14+100 to 14+140. Shrinkage cracks from Sta. 14+200 to 14+350.	Inboard slope: At bulged toe and slip locations, excavate trapezoid equal to equipment width parallel to slope face from toe to hinge point; replace with engineered fill. Outboard slope: At crack locations, remove top 450 mm of levee slope full height of slope and replace with engineered fill.
	Settlement				None
	Erosion	Eroded at chain-link fence location under bridge near Sta. 13+710. Scour at toe between Sta. 14+225 and 14+330.			Fill scoured area at toe with engineered fill.

Note 1. Station locations listed are approximate values based on visual estimates.
Note 2. Conditions were observed on December 19, 22 and 23, 1997.

TABLE 9 (continued)
SITE RECONNAISSANCE SUMMARY OF EAST LEVEE
December 19, 22, and 23, 1997
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT
95-973017NB.00

Station	Observed Condition Category	Location on Surface of Levee [1, 2]			
		Inboard Slope	Crest	Outboard Slope	Anticipated Repair
13+650 to 14+500 (continued)	Others				None
Reach A (From the downstream side of Highway 101 to Station 15+510)					
14+500 to 15+510	Burrow Holes				None
	Seepage				None
	Slip/Crack	Steep-sided old access ramp (1m to 1½m drop at toe) from Sta. 14+780 to 14+890. 1m drop at toe from Sta. 14+980 to 15+100; ¾m drop from Sta. 15+100 to 15+300.			Inboard slope: Flatten toe of slope between Sta. 14+780 to 14+890, 14+980 to 15+100 and 15+100 to15+300. Replace with engineered fill and match slope of adjacent areas.
	Settlement		Tire rut from Sta. 15+390 to 15+510.		On full width of crest excavate top 460 mm and replace (tire rut) with engineered fill between Sta. 15+390 to 15+510.
	Erosion	Scour at toe forming gullies between Sta. 14+710 and 14+725.			Fill gullies with engineered fill.
	Others	Trees near Sta. 14+780 and 15+025.			Remove trees and stumps and backfill stump hole with engineered fill.

Note 1. Station locations listed are approximate values based on visual estimates.
Note 2. Conditions were observed on December 19, 22 and 23, 1997.

TABLE 10

SITE RECONNAISSANCE SUMMARY OF WEST LEVEE
December 19, 22, and 23, 1997
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT
95-973017NB.00

Station	Observed Condition Category	Location on Surface of Levee[1, 2]			Anticipated Repair
		Inboard Slope	Crest	Outboard Slope	
Reach G (From the downstream side of SPRR bridge to downstream side of Highway 237)					
7+000 to 8+000	Burrow Holes	Near Sta. 7+950			Backfill burrow holes.
	Seepage			Wet spot at toe near Sta. 7+630	None
	Slip/Crack				None
	Settlement		Sagged between Sta. 7+880 and Sta. 7+950. Hummocky between Sta. 7+960 and Sta. 8+000.		Raise grade with engineered fill in sagged and hummocky areas.
	Erosion				None
	Others				None
Reach F (From the downstream side of Highway 237 to downstream side of Tasman Drive)					
8+000 to 9+200	Burrow Holes	Near Sta. 8+080. Various holes between Sta. 8+140 and Sta. 8+250. Near Sta. 8+500, Sta. 8+550, cluster at Sta. 9+070.		Near Sta. 9+130.	Backfill burrow holes.
	Seepage				None
	Slip/Crack	Uncompacted to poorly compacted edge near crest and/or various discontinuous cracks along upper slope between Sta. 8+200 and Sta. 9+150.			Inboard Slope: Between Sta. 8+200 and 9+150, excavate top 300 mm of levee fill on crest for half width. Replace with engineered fill. Outboard Slope: Between Sta. 8+200 and 8+500, estimated factor of safety (FOS) less than minimum FOS required. Flatten slope to increase FOS. Final slope will depend on finished raised crest grade.
	Settlement	Some sagging between Sta. 8+400 and 8+700.	Hummocky near inboard edge between Sta. 8+400 and Sta. 9+000. Potholes near Sta. 8+890. Uncompacted edge, longitudinal cracks and 12mm sag near Sta. 9+010.		Raise grade with engineered fill in sag between Sta. 8+400 and Sta. 9 +000. Excavate pothole to depth of 450 mm at Sta. 8+890 and replace with engineered fill. Near Sta. 9+010 excavate top 300 mm of levee fill on crest for half width on inboard side for a distance of 10 meters; replace with engineered fill.
	Erosion	Rill from Sta. 8+400 to Sta. 8+550. Some gully erosion near Sta. 8+610. Rill between Sta. 9+000 and Sta. 9+220.			Fill rills and gully with engineered fill.
	Others	Silted under Highway 237.			Hydraulic Engineer to review siltation under Highway 237.

Note.1. Station locations listed are approximate values based on visual estimates.
Note 2. Conditions were observed on December 19, 22 and 23, 1997.

TABLE 10 (continued)

Station	Observed Condition Category	Location on Surface of Levee[1, 2]			Anticipated Repair
		Inboard Slope	Crest	Outboard Slope	
Reach E (From the downstream side of Tasman Drive to downstream side of Hetch Hetchy)					
9+200 to 10+100	Burrow Holes	Near Sta. 9+490 and 9+930.		Near Sta. 9+290.	Backfill burrow holes.
	Seepage	Wet slope face near Sta. 9+400.			None
	Slip/Crack	Surface cracks from toe to crest near Sta. 9+750.		Steep side slope from Sta. 9+760 to 9+850.	Both inboard and outboard slopes: Between Sta. 9+230 and 10+100, estimated factor of safety (FOS) less than minimum FOS required. Flatten slope to increase FOS. Final slope will depend on finished raised crest grade.
	Settlement		Tire rut near Sta. 9+280 Tire rut near edge at Sta. 9+450. Tire rut from Sta. 9+950 to 10+000.		On half width of crest excavate top 460 mm and replace (tire ruts) with engineered fill between Sta. 9+270 to 9+290, 9+440 to 9+460 and 9+950 to 10+000.
	Erosion	Gully erosion from Sta. 9+650 to 9+700. Sheet erosion from Sta. 9+700 to 9+900		Sheet erosion along upper edge of slope near Sta. 9+500. Sheet erosion near Sta. 9+720. Sheet erosion in upper 3m from Sta. 9+780 to 9+830. Gully erosion due to bike tracks at Sta. 10+000.	Inboard slope; fill gullies eroded between Sta. 9+650 to 9+700. Outboard slope: Fill gully at Sta. 10+000.
	Others				None
Reach D (From the downstream side of Hetch Hetchy to downstream side of Montague Expressway)					
10+100 to 11+800	Burrow Holes	At Sta. 10+130, 10+530, 11+210, 11+220 and 11+790. Cluster at Sta. 11+470.		At Sta. 11+750.	Backfill burrow holes.
	Seepage	300 mm outfall pipe located mid height of levee (unknown source) at Sta. 10+600. Plugged monitoring well at Sta. 10+890. Locked well at Sta. 11+010.			Survey profile and alignment of outfall pipe at Sta. 10+600; determine if it should be plugged or removed. District to determine if wells at Sta. 10+890 and 11+010 are still active; if inactive District should abandon them in accordance with District standards.
	Slip/Crack	Crack at Sta. 11+500. 2m longitudinal crack with ½ m wide bare strip at Sta. 11+695.		8m long crack near edge at Sta. 10+105. Grading contractor scraped off slightly 1m below crest from Sta. 10+400 to 10+470. Over-steepened side slope of 1995 lift from Sta. 11+200 to 11+400. 60m longitudinal tension crack from Sta. 11+660 to 11+720. Longitudinal crack from Sta. 11+745 to 11+790	Both inboard and outboard slopes: where crack is located near hinge point, excavate top 300 mm of levee fill for half crest width. Replace with engineered fill. Where crack is located on slope face, excavate trapezoid equal to equipment width parallel to slope face from crest to toe. Replace with engineered fill. Outboard slope: Between Sta. 10+400 to 10+470 place engineered fill.
	Settlement				None

Note.1. Station locations listed are approximate values based on visual estimates.
Note 2. Conditions were observed on December 19, 22 and 23, 1997.

TABLE 10 (continued)

ation	Observed Condition Category	Location on Surface of Levee[1,2]			Anticipated Repair
		Inboard Slope	Crest	Outboard Slope	
10+100 to 11+800 (continued)	Erosion	Gully erosion at Sta. 10+420. Sheet erosion from Sta. 10+430 to 10+500. Gully erosion at Sta. 10+535. Sheet erosion from Sta. 10+740 to 10+760. Scour at toe from Sta. 10+890 to 10+903 (3m high) and from Sta. 10+980 to 11+000 (5m high). Gulley at Sta. 11+470 and Sta. 11+670.		Sheet erosion near Sta. 10+180. Disturbed slope face from Sta. 10+500 to 10+506.	Inboard slope: Fill gullies at Sta. 10+420, 10+535, 11+470, and 11+670 with engineered fill. Fill scour at both toe areas from Sta. 10+890 to 10+903 and from Sta. 10+980 to 11+000 with engineered fill; match adjacent grades. Outboard slope: Between Sta. 10+500 and 10+506, recompact disturbed slope face.
	Others	Tree near Sta. 11+325		Lick Mill property fence from Sta. 10+930 to 11+650. Vegetation and irrigation lines at toe from Sta. 10+930 to Sta. 11+650.	Remove tree and stump; backfill stump hole with engineered fill.
Reach C (From the downstream side of Montague Expressway to downstream side of Trimble Road)					
11+800 to 13+650	Burrow Holes	At Sta. 12+590 and 12+660.		At Sta. 13+540.	Backfill burrow holes.
	Seepage	Blocked weep holes on concrete slope face from Sta. 11+870 to 11+900.			Unplug blocked weep holes on concrete slope face.
	Slip/Crack	1m drop at toe from inner shelf from Sta. 11+940 to 11+960, and from Sta. 11+990 to 12+010. 1m drop near toe from Sta. 12+300 to 12+455. Steep side slope along old maintenance road from Sta. 12+755 to 12+765. 5m longitudinal crack at Sta. 12+845.		Slightly bulged at toe and pushed fence post outward between Sta. 12+745 and 12+770. Bulged at toe near Sta. 12+860.	Inboard slope: Flatten toe of lower slope with engineered fill between Sta. 11+940 to 11+960, 11+990 to 12+010, 12+300 to 12+455; match adjacent grades. Where crack is located on slope face, excavate trapezoid equal to equipment width parallel to slope face from crest to toe. Replace with engineered fill. Outboard slope: Between Sta. 12+745 and 12+770, excavate trapezoid equal to equipment width parallel to slope face from crest to toe. Replace with engineered fill.
	Settlement	Soft spots at toe between Sta. 13+365 and 13+375.	Sag at Sta. 13+340.		Raise grade in sag with engineered fill.
	Erosion		Sheet erosion between Sta. 12+240 and 12+260		None
	Others	Trees near Sta. 12+225, 12+275 and between 12+300 and 12+800		Access ladder cut into slope at Sta. 12+280. Trees near Sta. 12+225,12+275 and between 12+300 and 12+550.	Remove trees and tree stumps; backfill stump holes with engineered fill. Remove access ladder at Sta. 12+280 on outboard slope and backfill excavation with engineered fill.

ote.1. Station locations listed are approximate values based on visual estimates.
Note 2. Conditions were observed on December 19, 22 and 23, 1997.

TABLE 10 (continued)

Station	Observed Condition Category	Location on Surface of Levee[1, 2]			Anticipated Repair
		Inboard Slope	Crest	Outboard Slope	
Reach B (From the downstream side of Trimble Road to downstream side of Highway 101)					
13+650 to 14+500	Burrow Holes	At Sta. 13+725 and 13+825.			Backfill burrow holes.
	Seepage				None
	Slip/Crack	Shrinkage cracks from Sta. 13+800 to 14+275. Disturbed slope surface from tree removal between Sta. 14+400 and 14+410. Steep side slope (>1:1) from Sta. 14+420 to 14+450.		Shrinkage cracks from Sta. 13+800 to 14+275.	Inboard and Outboard slope: Between Sta. 13+800 to 14+275 excavate trapezoid equal to equipment width parallel to slope face from crest to toe. Replace with engineered fill. Inboard slope: Between Sta. 14+400 to 14+410, excavate trapezoid equal to equipment width parallel to slope face from crest to toe. Between Sta. 14+420 to 14+450 place engineered fill and flatten slope to match adjacent slopes.
	Settlement	Cracks around concrete conduit near Sta. 14+010; possible settlement of foundation.	Tire rut between Sta. 13+910 and 13+940. Tire rut and shrinkage cracks between Sta. 14+100 and 14+275.		Structural Engineer to inspect cracks at concrete conduit at Sta. 14+010 and determine need for repairs, if any. On full width of crest excavate top 600 mm and replace (tire ruts) with engineered fill between Sta. 13+910 to 13+940 and Sta. 14+100 to 14+275.
	Erosion	Minor piping underneath bridge abutment at Sta. 13+690.			None
	Others	Occasional large trees from Sta. 14+100 to 14+400.		Tree near Sta. 14+015	Remove trees and tree stumps; backfill stump holes with engineered fill.
Reach A (From the downstream side of Highway 101 to downstream side of I880)					
14+500 to 17+550	Burrow Holes	At Sta. 15+770.	At Sta. 16+040.		Backfill burrow holes.
	Seepage				None
	Slip/Crack	Tension crack near crest at Sta. 14+555 Shallow slip at toe and cracks on concrete sidewalk at Sta. 14+595. Rubble fill on steep slope (>1:1) at Sta. 14+585. Tension crack at Sta. 14+790. Slope change with steep drop at toe beginning at Sta. 14+820. Soft bank at Sta. 15+025. Slope change with steep drop at toe between Sta. 15+130 and 15+140. Slope change with steep drop at toe between Sta. 15+360 and 15+375. Slope change with steep drop (5m from crest) at toe between Sta. 15+500 and 15+510.	Uncompacted fill at Sta. 15+630, possibly due to disposal of construction debris resulting from nearby new power pole construction.	Longitudinal cracks along roadway (asphalt patched) from Sta. 15+190 to 15+360.	Inboard slope: At tension crack locations at Sta. 14+555, 14+790, between Sta. 15+700 to 15+765, and between Sta. 16+160 to 16+240, excavate trapezoid equal to equipment width parallel to slope face from crest to 3 meters below hinge point. Replace with engineered fill. Flatten toe of slope by placing engineered fill at steep drops beginning at Sta. 14+820, between 15+130 to 15+140, 15+360 to 15+375, 15+500 to 15+510; match with adjacent slopes. Flatten drops by placing engineered fill at Sta. 16+800, between 16+890 to 17+125, at 17+230, 17+260 and between 17+340 to 17+525; match with adjacent slopes. Between Sta. 15+700 to 15+765, 16+460 to 16+480, and 17+560 to 17+580, excavate trapezoid equal to equipment width parallel to slope face from crest to halfway down to toe; replace with engineered fill. Between Sta. 15+630 to 15+660, excavate trapezoid equal to equipment width parallel to slope from hinge point to toe and replace with engineered fill.

Note.1. Station locations listed are approximate values based on visual estimates.
Note 2. Conditions were observed on December 19, 22 and 23, 1997.

TABLE 10 (continued)

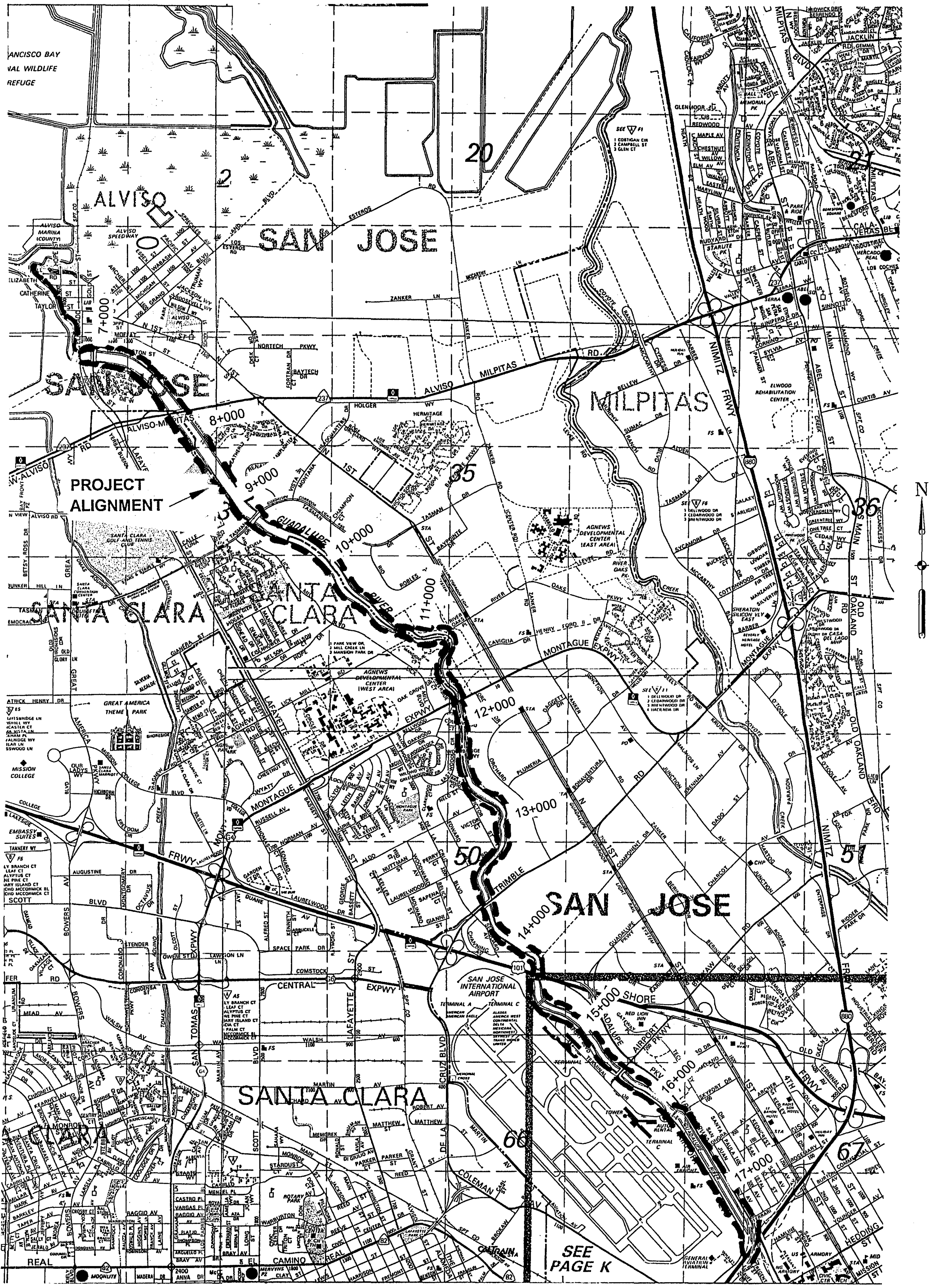
Station	Observed Condition Category	Location on Surface of Levee[1, 2]			Anticipated Repair
		Inboard Slope	Crest	Outboard Slope	
14+500 to 17+550 (continued)	Slip/Crack	Shallow slip and tension crack between Sta. 15+700 and 15+765. Steep-sided access road at Sta. 15+780. Tension cracks near crest from Sta. 16+160 to 16+240. Shallow slip with cracks and slump at Sta. 16+470. 2m cut with steep slope at Sta. 16+800. Slope change with 1 to 2m drop from Sta. 16+890 to Sta. 17+125. 4m drop at Sta. 17+230. 2m drop at Sta. 17+260. 6m drop from crest from Sta. 17+340 to 17+525. Shallow slip from Sta. 17+560 to 17+580.			Inboard slope: At tension crack locations at Sta. 14+555, 14+790, between Sta. 15+700 to 15+765, and between Sta. 16+160 to 16+240, excavate trapezoid equal to equipment width parallel to slope face from crest to 3 meters below hinge point. Replace with engineered fill. Flatten toe of slope by placing engineered fill at steep drops beginning at Sta. 14+820, between 15+130 to 15+140, 15+360 to 15+375, 15+500 to 15+510; match with adjacent slopes. Flatten drops by placing engineered fill at Sta. 16+800, between 16+890 to 17+125, at 17+230, 17+260 and between 17+340 to 17+525; match with adjacent slopes. Between Sta. 15+700 to 15+765, 16+460 to 16+480, 16+790 to 16+810 and 17+560 to 17+580, excavate trapezoid equal to equipment width parallel to slope face from crest to 1m below top of crack; replace with engineered fill. Between Sta. 15+630 to 15+660, excavate trapezoid equal to equipment width parallel to slope from hinge point to toe and replace with engineered fill.
	Settlement	Hummocky at Sta. 14+790.	Tire rut from Sta. 14+820 to 14+930. Hummocky from Sta. 14+790 to 14+930. Hummocky from Sta. 15+300 to 15+400. Tire rut between Sta. 15+700 and 15+720. Tire rut at Sta. 16+030. Tire rut between Sta. 16+500 and 16+700. Hummocky at Sta. 16+805 and 16+855. Tire rut at Sta. 16+995.		On full width of crest excavate top 460 mm and replace (tire ruts) with engineered fill between Sta. 14+820 to 14+930, 15+700 to 15+720, 16+020 to 16+040, 16+500 to 16+700, and 16+985 to 17+005. Raise grade in hummocky areas with engineered fill.
	Erosion	Slight undermine of concrete and fill interface at Sta. 14+710. Sheet erosion at Sta. 14+790. Scour under SCSP at Sta. 15+500. Scour at toe between Sta. 15+560 and 15+580. Scour around SCSP at Sta. 15+600. Scour at toe between Sta. 15+590 and 15+610. Some undermining of SCSP at Sta. 15+700. Disturbed creek bank due to construction equipment between Sta. 16+240 and		Gully erosion at Sta. 16+640.	Inboard slope: Place additional SCSP in scoured/undermined areas adjacent to existing SCSP and at abandoned structure at Sta. 17+560. Fill in gullies with engineered fill. Between Sta. 16+240 to 16+250, excavate trapezoid equal to equipment width parallel to slope face from hinge point to toe; replace with engineered fill. Place riprap or SCSP at areas with undermining and/or scour at toe. Outboard slope: Fill in gully at Sta. 16+640 with engineered fill.

Note.1. Station locations listed are approximate values based on visual estimates.
Note 2. Conditions were observed on December 19, 22 and 23, 1997.

TABLE 10 (continued)

Station	Observed Condition Category	Location on Surface of Levee[1, 2]			Anticipated Repair
		Inboard Slope	Crest	Outboard Slope	
14+500 to 17+550 (continued)	Erosion	16+250. Gully erosional features at Sta. 16+580, 16+650 and 16+670. Gully at Sta. 16+890, 16+970, and sheet erosion at Sta. 17+040. Some undermine at Sta. 17+210. Heavy scour under concrete structure (abandoned) at Sta. 17+560.			
	Others	Buried concrete face channel, 300 mm slab at low flow line, concrete block of drop structure between Sta. 15+930 and 15+990. Miscellaneous concrete rubble on slope between Sta. 16+200 and 17+350. Trees near Sta. 15+025, 15+630, 15+680, 17+220 and 17+270.		Previously planted redwood trees between Sta. 15+670 and 16+200 were removed; however, it is unknown whether the root systems and tree stumps were excavated and removed.	Inboard slope: Remove trees and stumps and backfill with engineered fill. Outboard slope: Remove tree stumps between Sta. 15+670 to 16+200 and backfill stump holes with engineered fill.

Note. 1. Station locations listed are approximate values based on visual estimates.
Note 2. Conditions were observed on December 19, 22 and 23, 1997.



SCALE: 1"= 4260'

Project No. 95-9730717NB.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	**SITE MAP**	Figure 1
06/30/1999			
URS Greiner Woodward Clyde			

LEGEND:

- ◈ APPROXIMATE LOCATION OF EXPLORATORY BORING
- ▼ APPROXIMATE LOCATION OF CONE PENETRATION TEST
- APPROXIMATE LOCATION OF PREVIOUS BORING (SEE INDEX LIST ON TABLE 1)

NOTES:

1. PIEZOMETERS WERE INSTALLED IN BORINGS EB-4 EB-5 & EB-6.

SOURCE: CHANNEL ALIGNMENT IS BASED ON AUTOCAD DRAWING PROVIDED BY SCVWD.

Q:\GUAD001.DWG

7/15/99

REV	DESCRIPTION OF REVISION	BY	DATE
1	MOVED LOCATION OF SEVERAL BORINGS AND CPTS AND REVISED NUMBERS.	S.H.	3/19/98
2	MOVED WC-4 AND WB-5 PER CULTURAL RESOURCE RECOMMENDATION.	S.H.	4/9/98

URS Greiner Woodward Clyde

55 South Market Street, Suite 1650
San Jose, Ca. 95113

WARNING
0 1/2 1
IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO SCALE

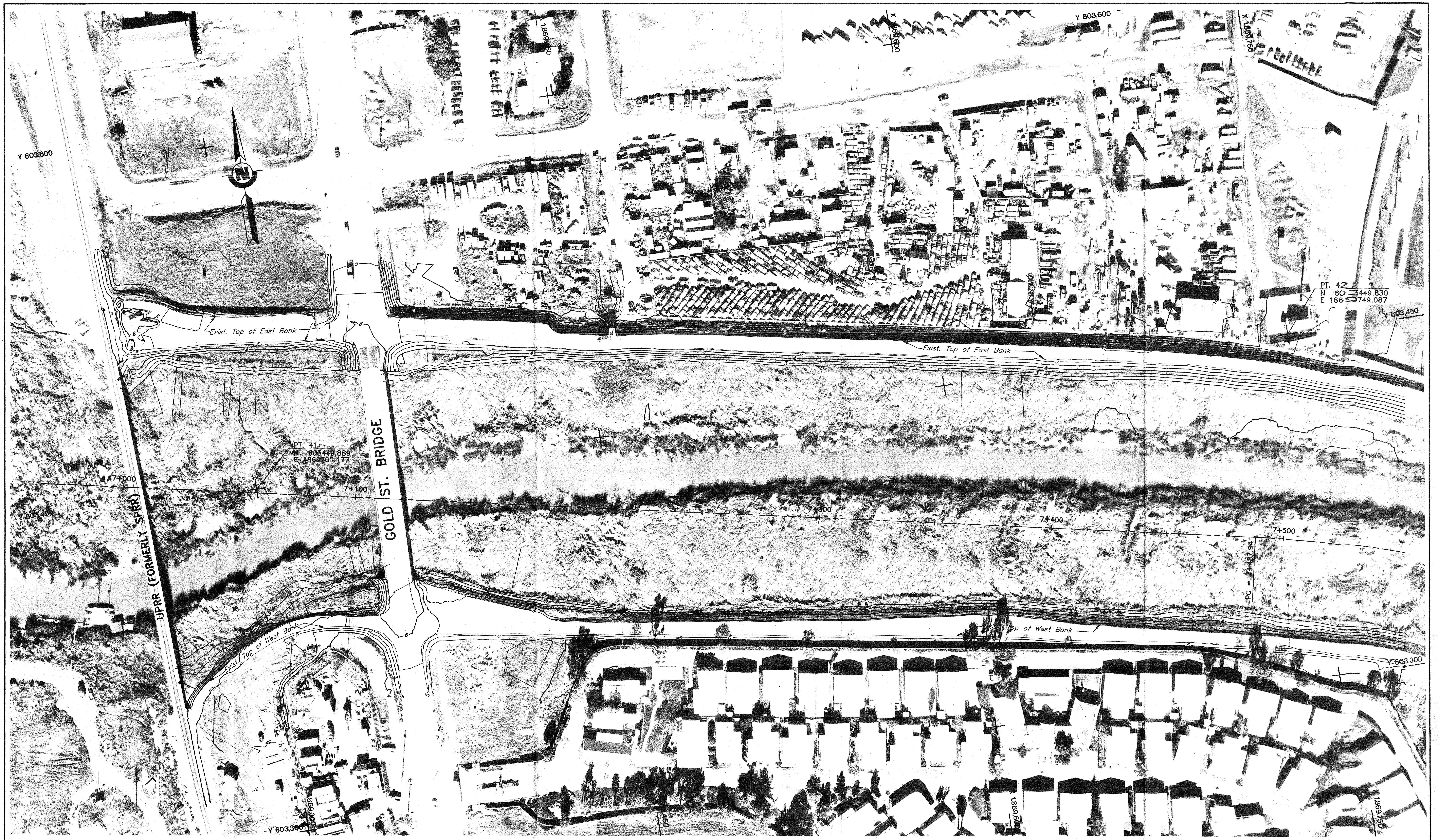
DESIGNED S.H.
DRAWN E.G.
CHECKED S.H.
PEER REVIEWED M.L.
PROJECT MANAGER P.B.
DATE 7/15/99

Lower Guadalupe River Flood Control Project Santa Clara County, CA.

FIELD EXPLORATION LOCATION PLAN
(PROJEXT NO. 301511)

REVISION 2
PROJECT 95-973017NB.00
DRAWING Fig 2
SHEET OF

TEMP013



REV	DESCRIPTION	DATE	APPR	REFERENCE INFORMATION AND NOTES

DATE 11-25-97

DESIGN

DRAWN

CHECKED

Santa Clara Valley Water District

ENGINEERING CERTIFICATION

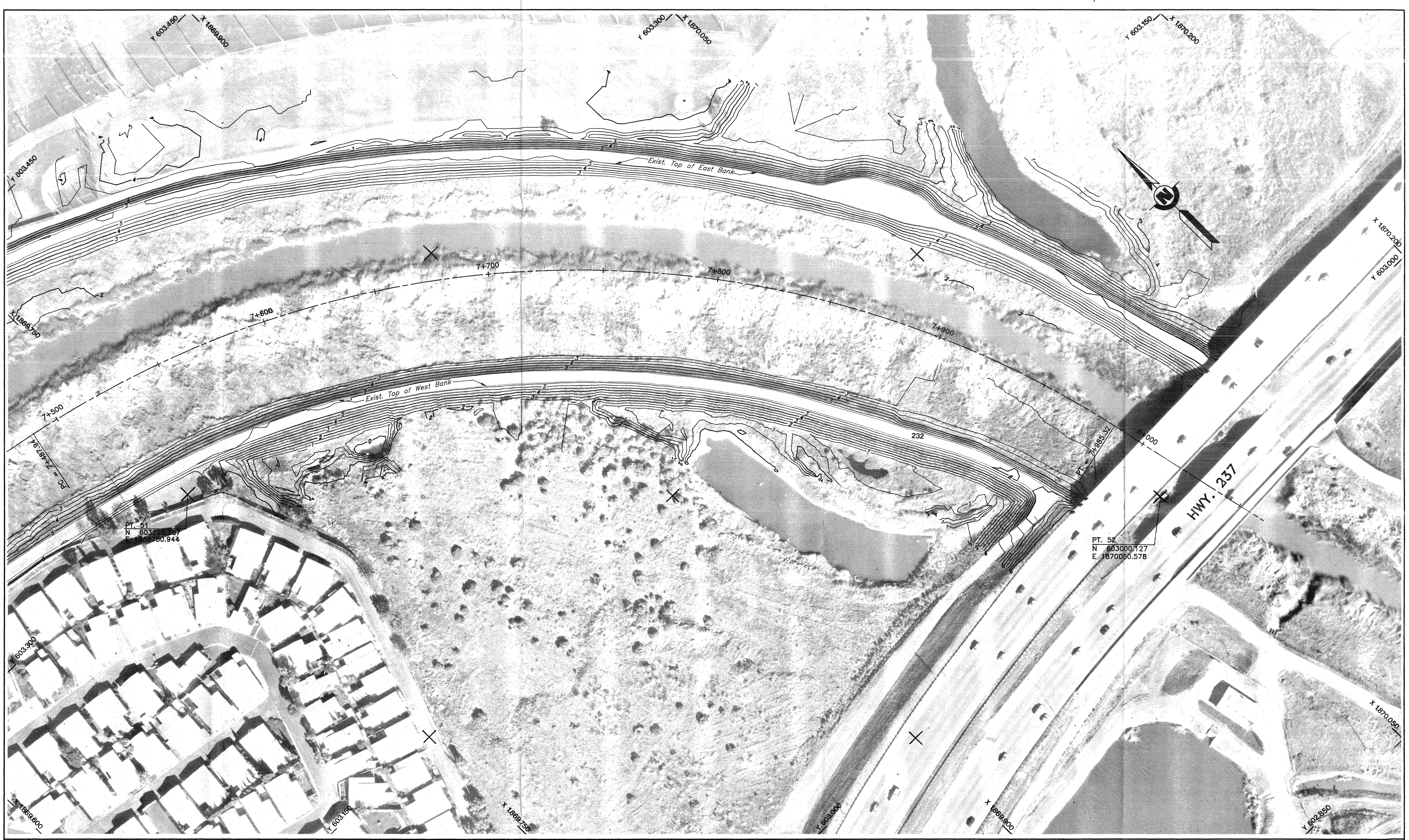
ENGINEERING APPROVAL

TOPOGRAPHIC MAP
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT
SANTA CLARA COUNTY, CALIFORNIA

SCALE
1 : 750 m

SHEET
Figure 3a

Aero-Geodetic Corp. 1-25-96 965710



REV	DESCRIPTION	DATE	APPR	REFERENCE INFORMATION AND NOTES

DATE 11-25-97
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Santa Clara Valley Water District

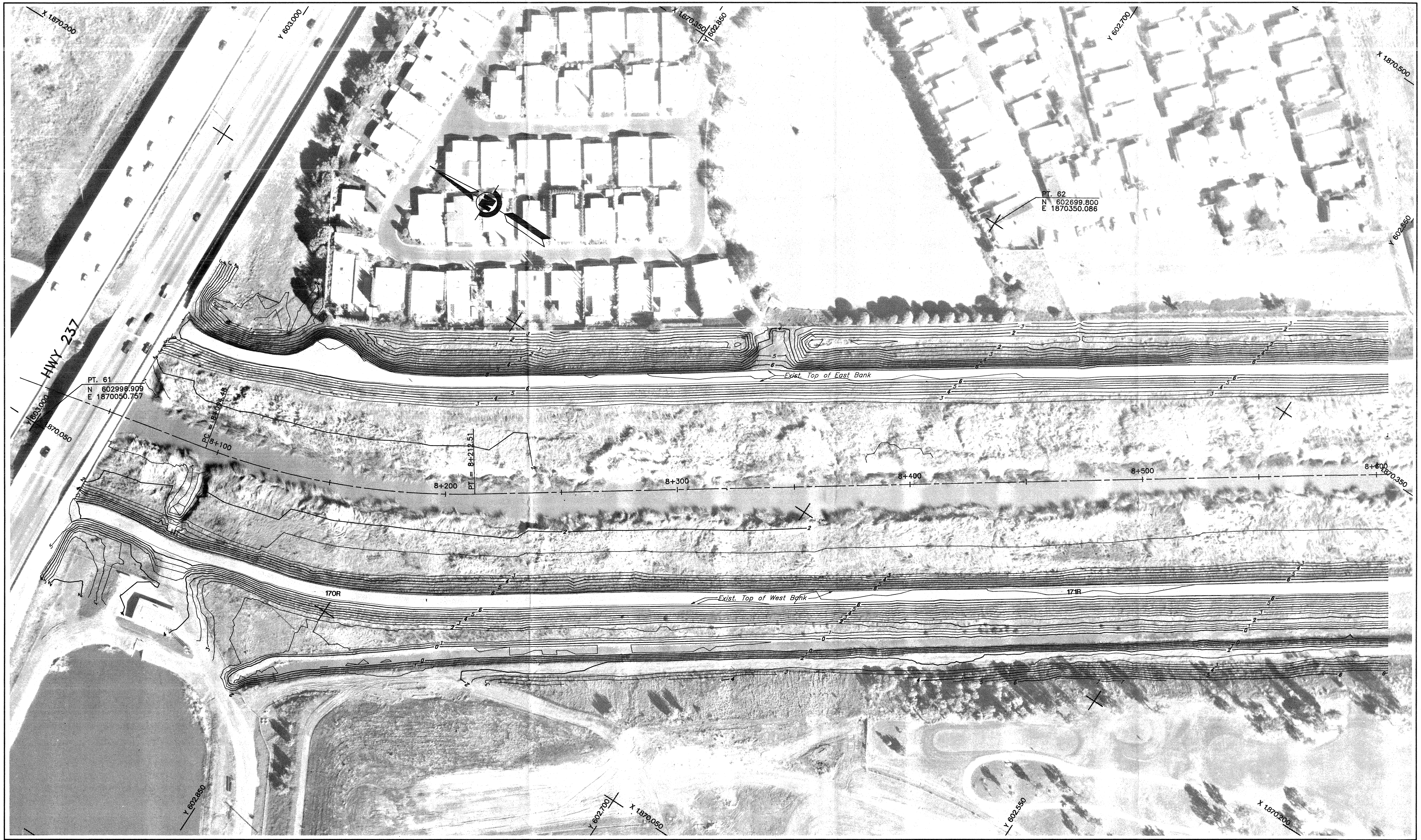
ENGINEERING CERTIFICATION
ENGINEERING APPROVAL

TOPOGRAPHIC MAP
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT
SANTA CLARA COUNTY, CALIFORNIA

SCALE 1 : 750 m

SHEET Figure 3b

Aero-Geodetic Corporation 1-25-96 965710



REV	DESCRIPTION	DATE	APPR	REFERENCE INFORMATION AND NOTES

DATE 11-25-97

DESIGN

DRAWN

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Santa Clara Valley Water District

ENGINEERING CERTIFICATION

ENGINEERING APPROVAL

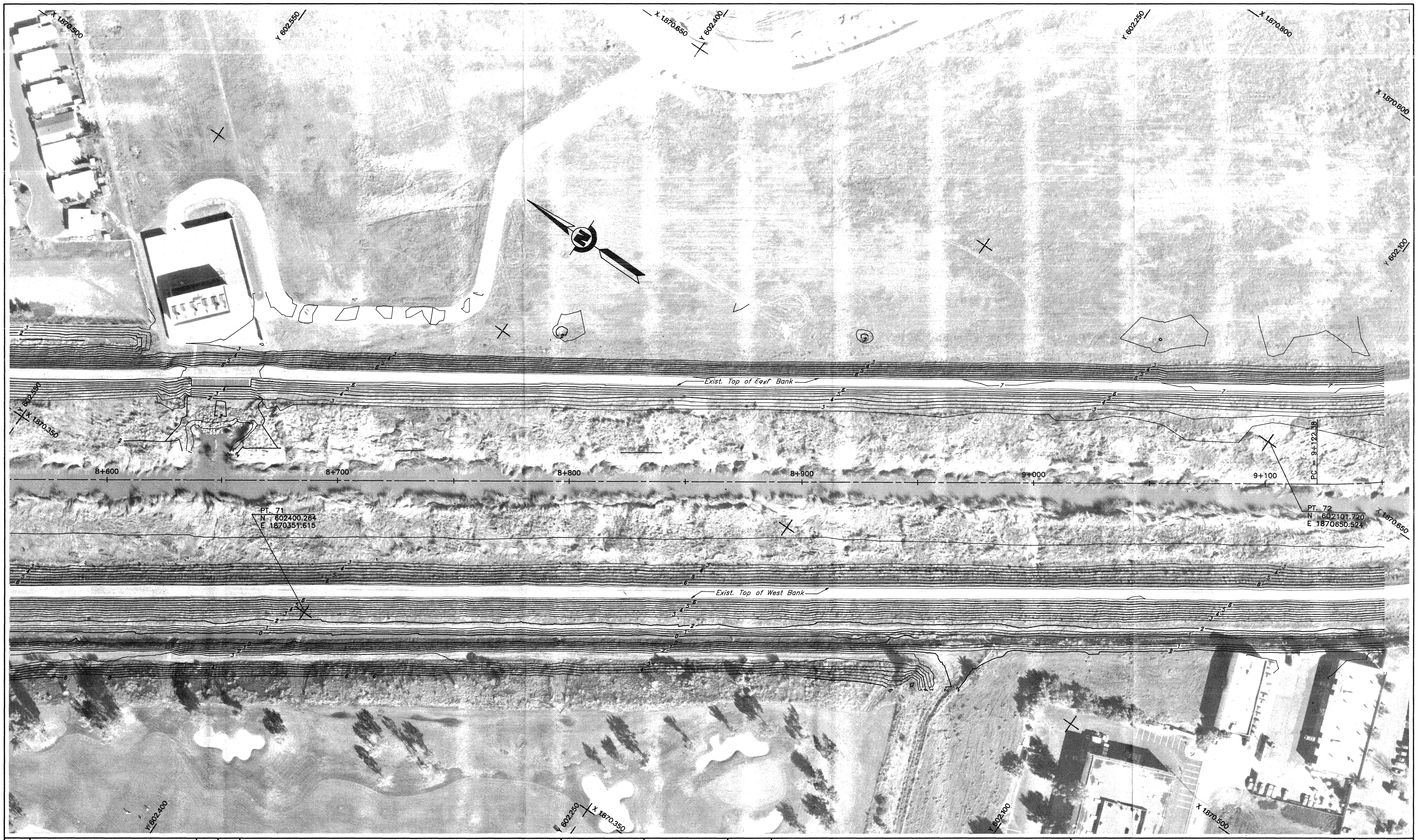
TOPOGRAPHIC MAP
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT
SANTA CLARA COUNTY, CALIFORNIA

SCALE
1 : 750 m

SHEET
Figure 3c

Aero-Geodetic Corporation 1-25-96 965710

3



REV	DESCRIPTION	DATE	APPR	REFERENCE INFORMATION AND NOTES

DATE 11-25-97

DESIGN

DRAWN

CHECKED

Santa Clara Valley Water District

ENGINEERING CERTIFICATION

ENGINEERING APPROVAL

TOPOGRAPHIC MAP
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT
SANTA CLARA COUNTY, CALIFORNIA

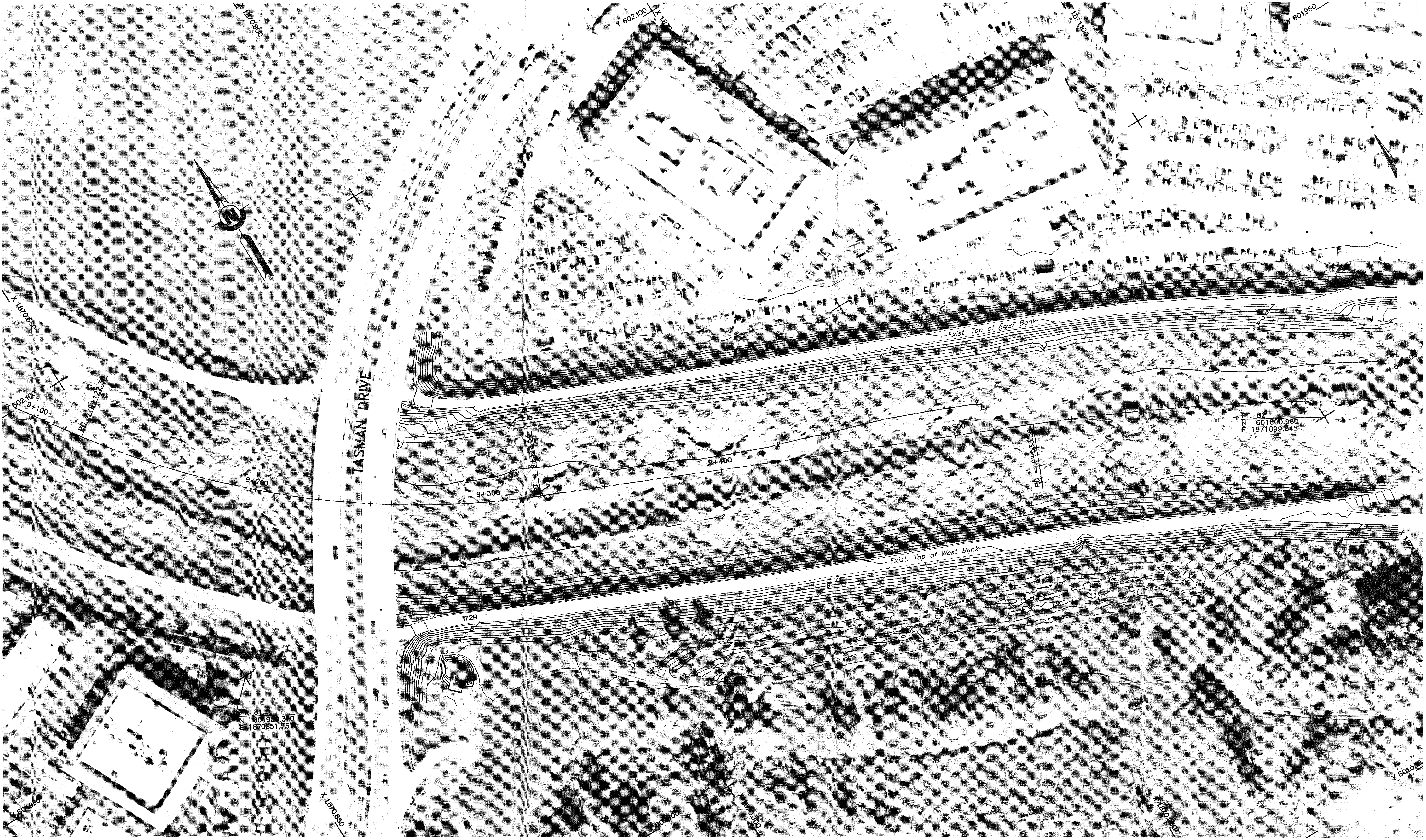
SCALE 1:750 m

SHEET

Figure 3d

Aero-Geodetic Corp. 1-25-96

4



REV	DESCRIPTION	DATE	APPR	REFERENCE INFORMATION AND NOTES

DATE 11-25-97
DESIGN
DRAWN
CHECKED

Santa Clara Valley Water District

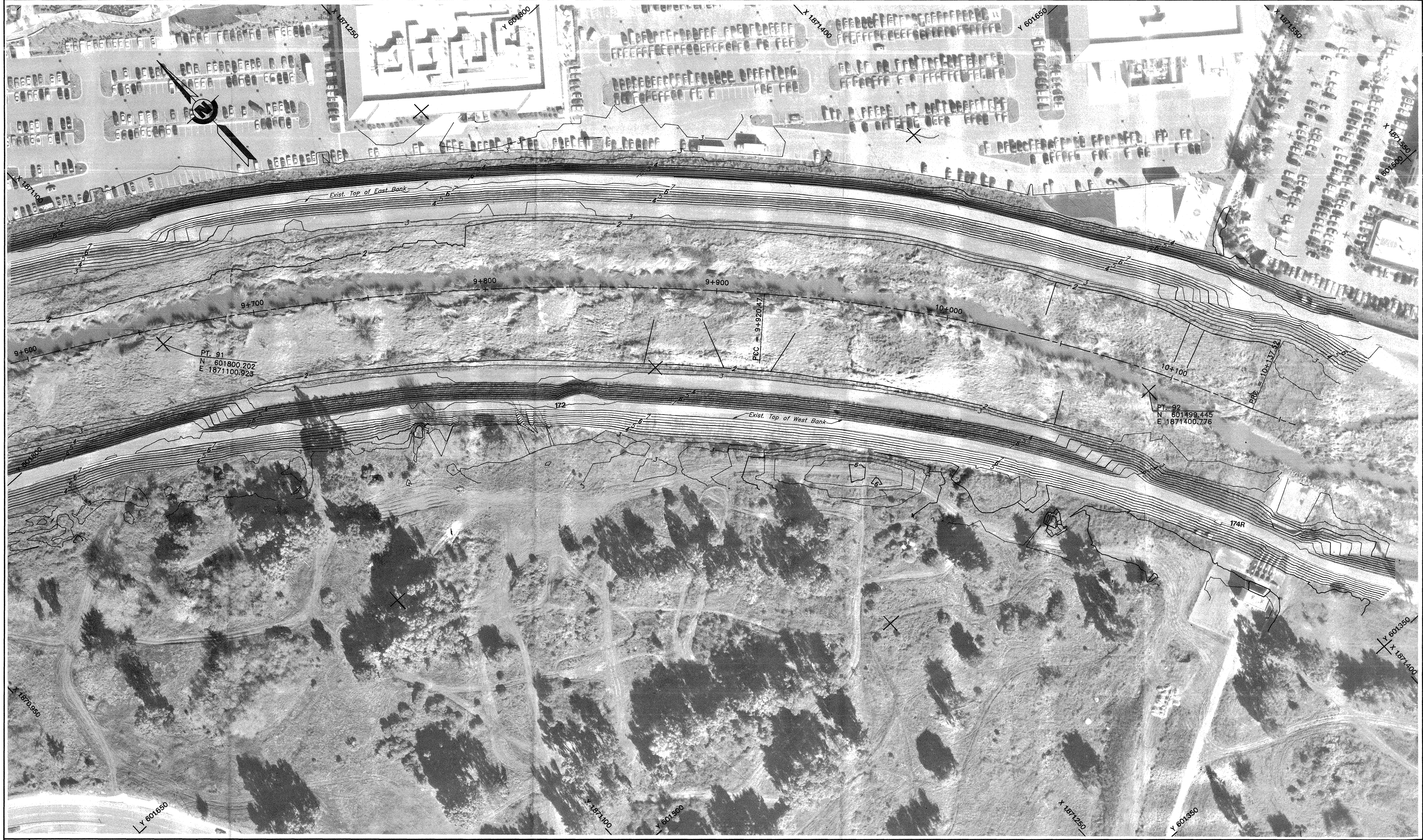
ENGINEERING CERTIFICATION
ENGINEERING APPROVAL

TOPOGRAPHIC MAP
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT
SANTA CLARA COUNTY, CALIFORNIA

SCALE
1 : 750 m

SHEET
Figure 3e

Aero-Geodetic Corporation 1-25-96 965710



REV	DESCRIPTION	DATE	APPR	REFERENCE INFORMATION AND NOTES

DATE 11-25-97

DESIGN

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Santa Clara Valley Water District

ENGINEERING CERTIFICATION

ENGINEERING APPROVAL

TOPOGRAPHIC MAP
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT
SANTA CLARA COUNTY, CALIFORNIA

SCALE
1 : 750 m

SHEET
Figure 3f

Aero-Geodetic Corporation 1-25-96 965710

REV	DESCRIPTION	DATE	APPR	REFERENCE INFORMATION AND NOTES

DATE
11·25·97
DESIGN
DRAWN
CHECKED

Santa Clara Valley Water District

ENGINEERING CERTIFICATION

ENGINEERING APPROVAL

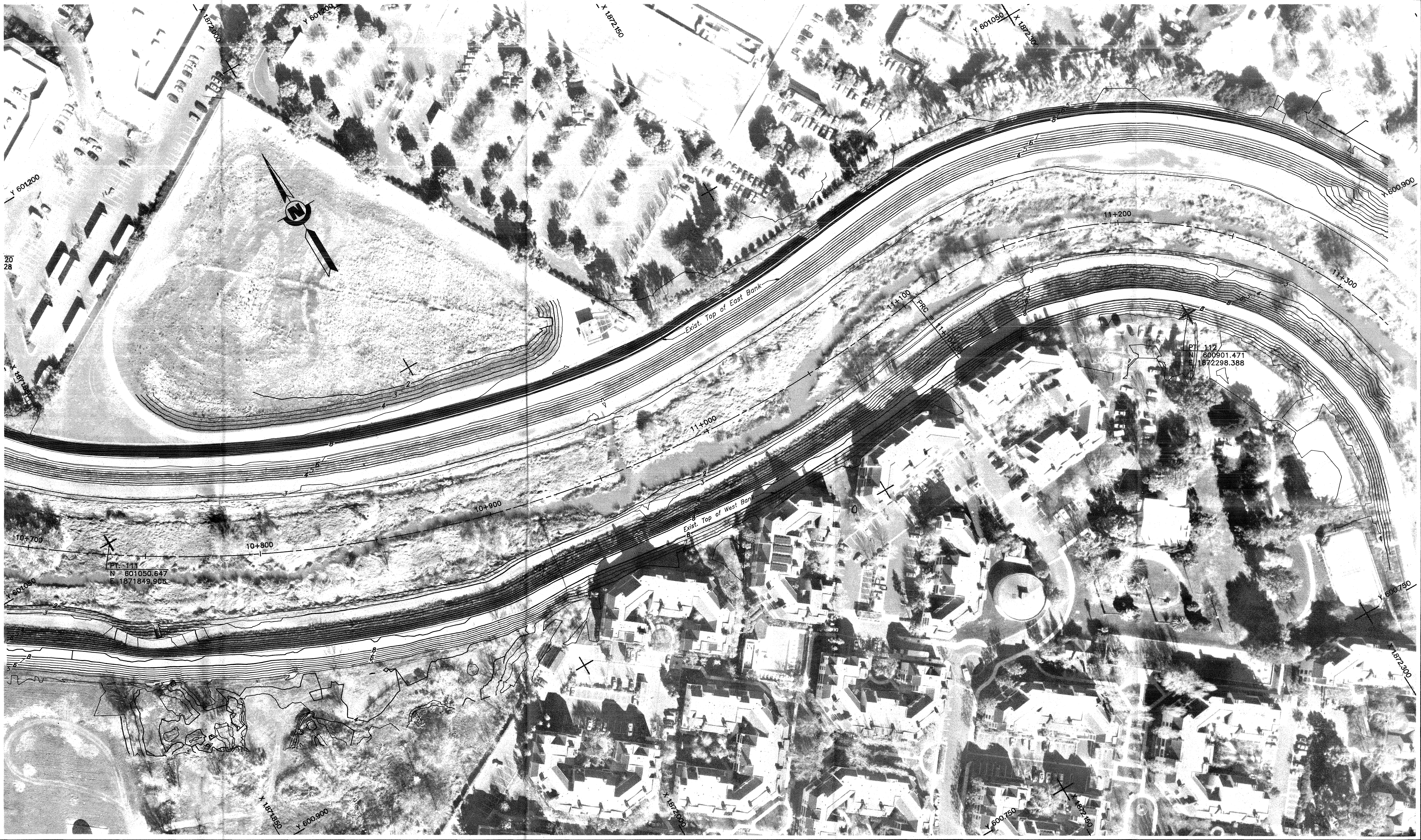
TOPOGRAPHIC MAP
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT
SANTA CLARA COUNTY, CALIFORNIA

SCALE
1 : 750 m

SHEET
Figure 3g

Aero Geodetic Corp. 2-1-96 965710

7



REV	DESCRIPTION	DATE	APPR	REFERENCE INFORMATION AND NOTES

DATE 11-25-97

DESIGN

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Santa Clara Valley Water District

ENGINEERING CERTIFICATION

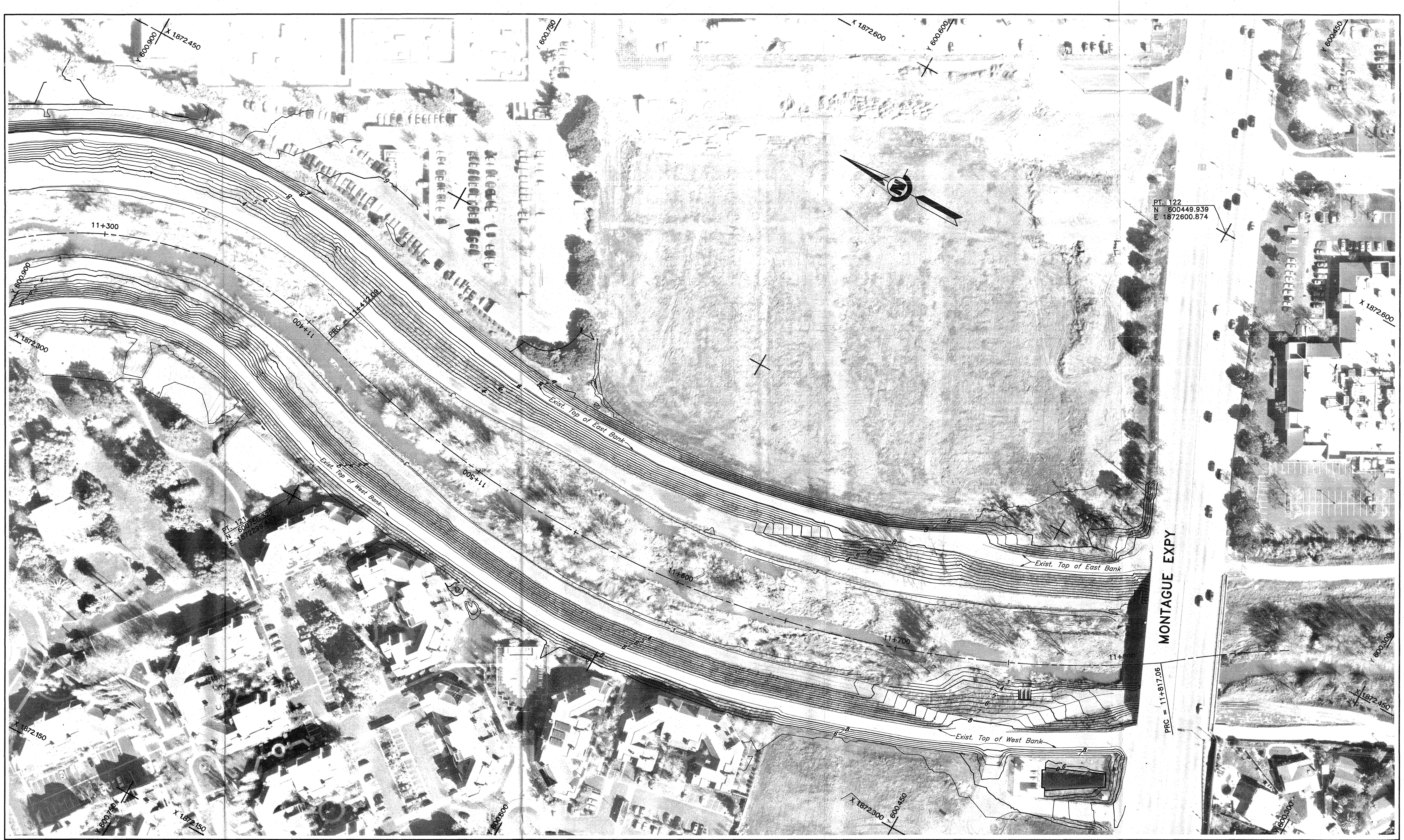
ENGINEERING APPROVAL

TOPOGRAPHIC MAP
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT
SANTA CLARA COUNTY, CALIFORNIA

SCALE 1 : 750 m

SHEET Figure 3h

Aero-Geodetic Corporation 1-25-96 965710



REV	DESCRIPTION	DATE	APPR	REFERENCE INFORMATION AND NOTES

DATE 11-25-97
DESIGN
DRAWN
CHECKED

Santa Clara Valley Water District

ENGINEERING CERTIFICATION
ENGINEERING APPROVAL

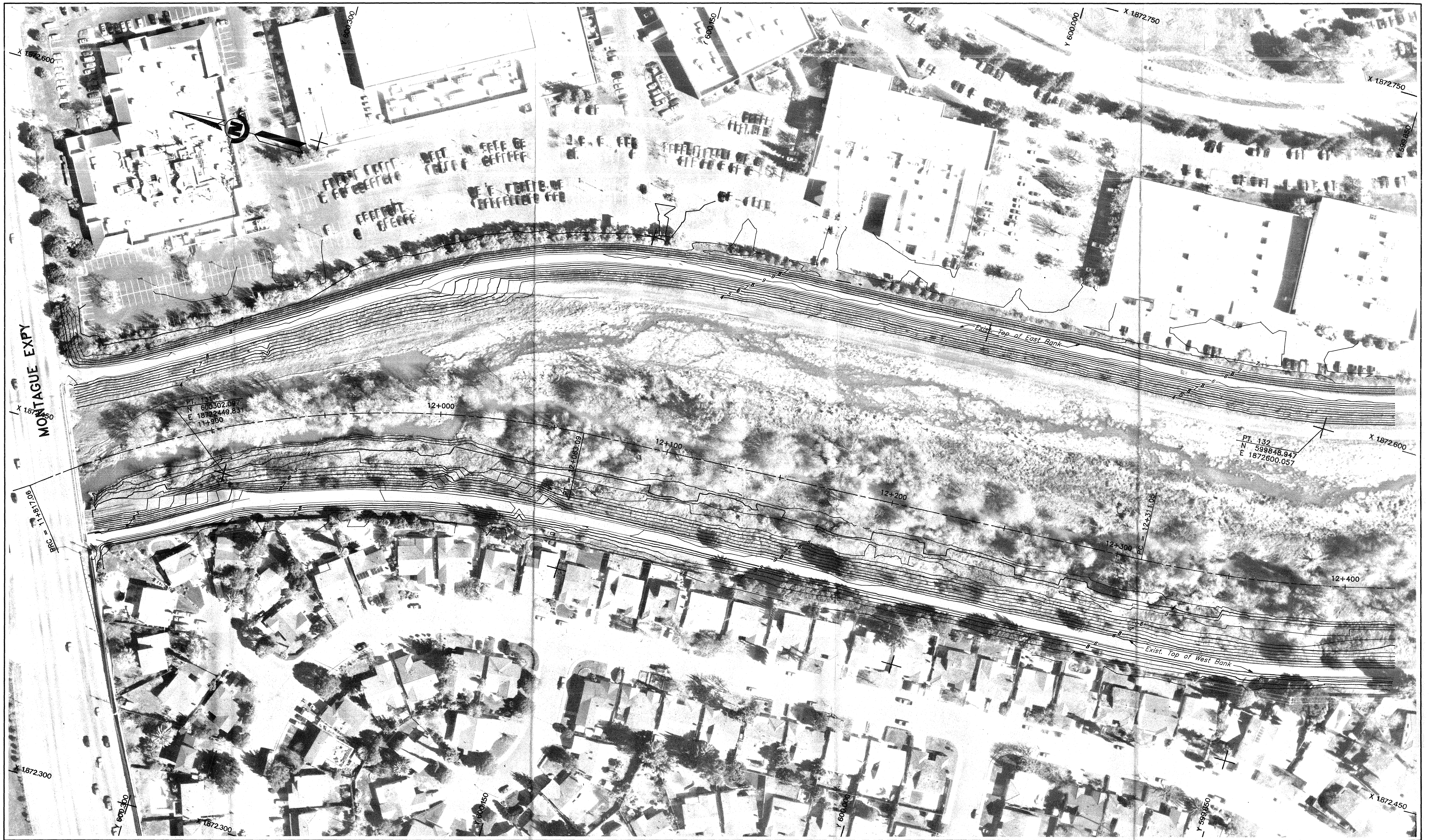
TOPOGRAPHIC MAP
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT
SANTA CLARA COUNTY, CALIFORNIA

SCALE
1 : 750 m

SHEET
Figure 3i

Aero-Geodetic Corp. 1-25-96 96[illegible]

9



REV	DESCRIPTION	DATE	APPR

REFERENCE INFORMATION AND NOTES

DATE 11-25-97

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Santa Clara Valley Water District

ENGINEERING CERTIFICATION

ENGINEERING APPROVAL

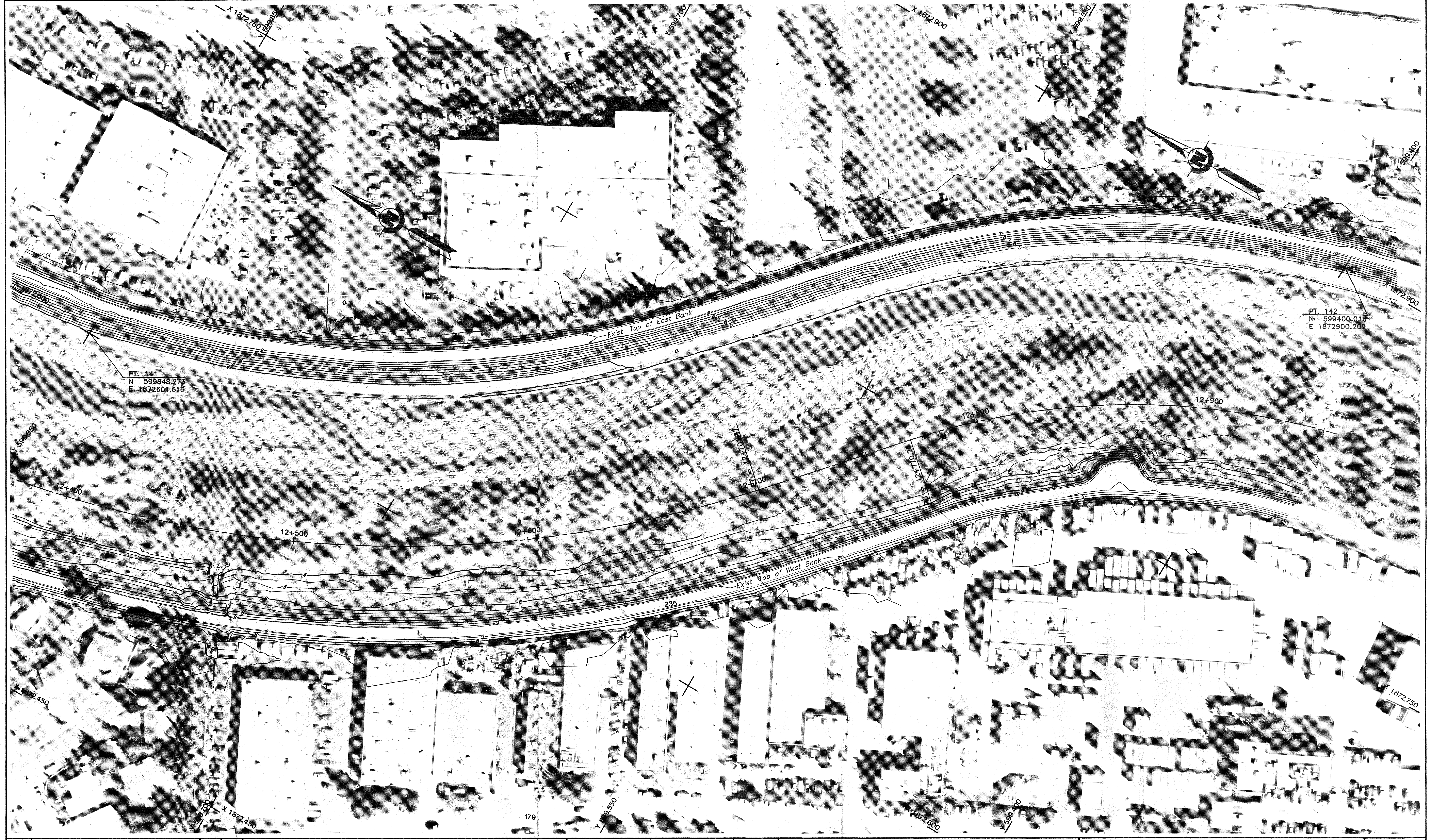
TOPOGRAPHIC MAP
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT
SANTA CLARA COUNTY, CALIFORNIA

SCALE
1 : 750 m

SHEET

Figure 3j

Aero-Geodetic Corporation 1-25-96 965710



REV	DESCRIPTION	DATE	APPR	REFERENCE INFORMATION AND NOTES

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Santa Clara Valley Water District

ENGINEERING CERTIFICATION

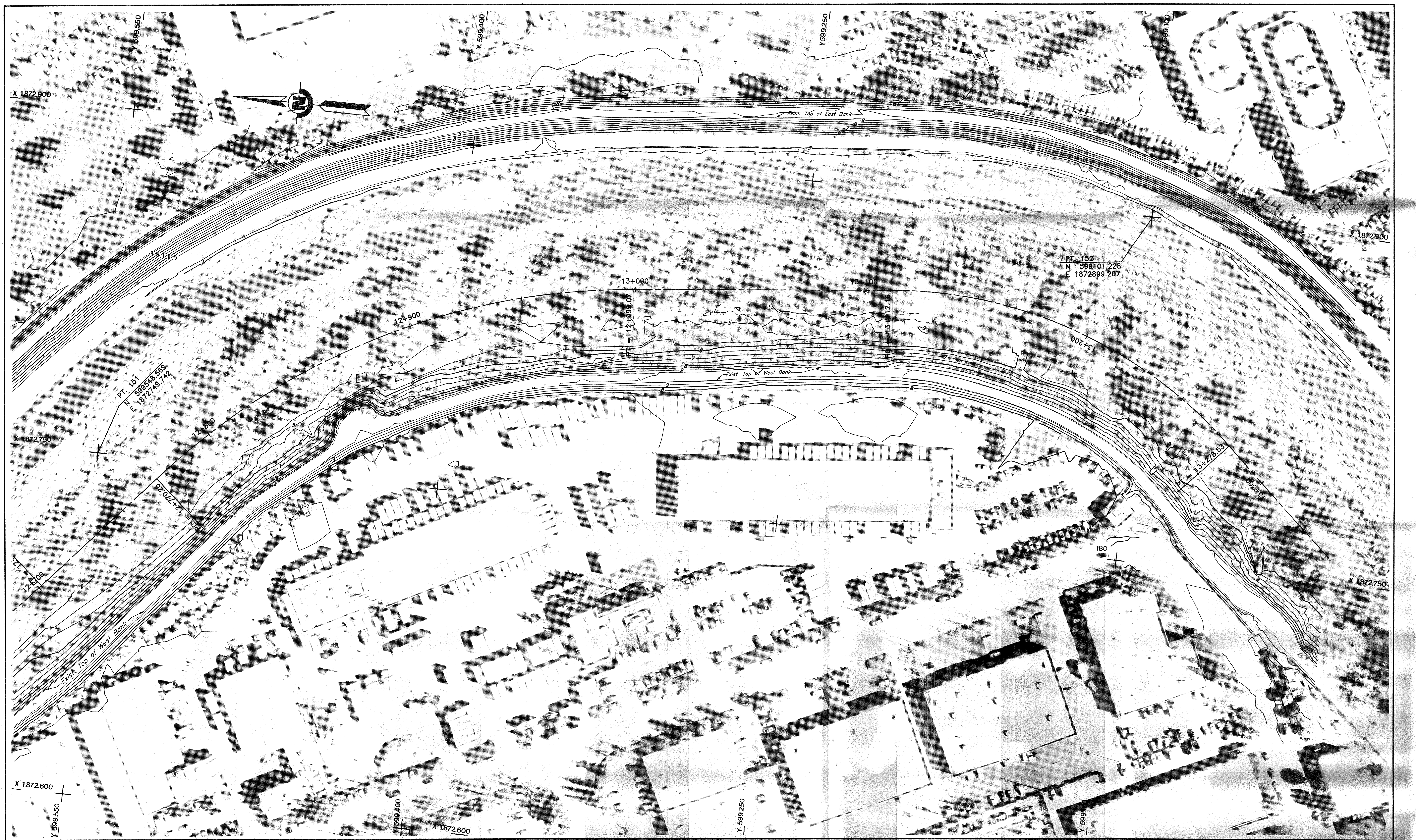
ENGINEERING APPROVAL

TOPOGRAPHIC MAP
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT
SANTA CLARA COUNTY, CALIFORNIA

SCALE
1 : 750 m

SHEET
Figure 3k

Aero-Geodetic Corp. 1-25-96 96



REV	DESCRIPTION	DATE	APPR	REFERENCE INFORMATION AND NOTES

DATE 11-25-97

DESIGN

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Santa Clara Valley Water District

ENGINEERING CERTIFICATION

ENGINEERING APPROVAL

TOPOGRAPHIC MAP
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT
SANTA CLARA COUNTY, CALIFORNIA

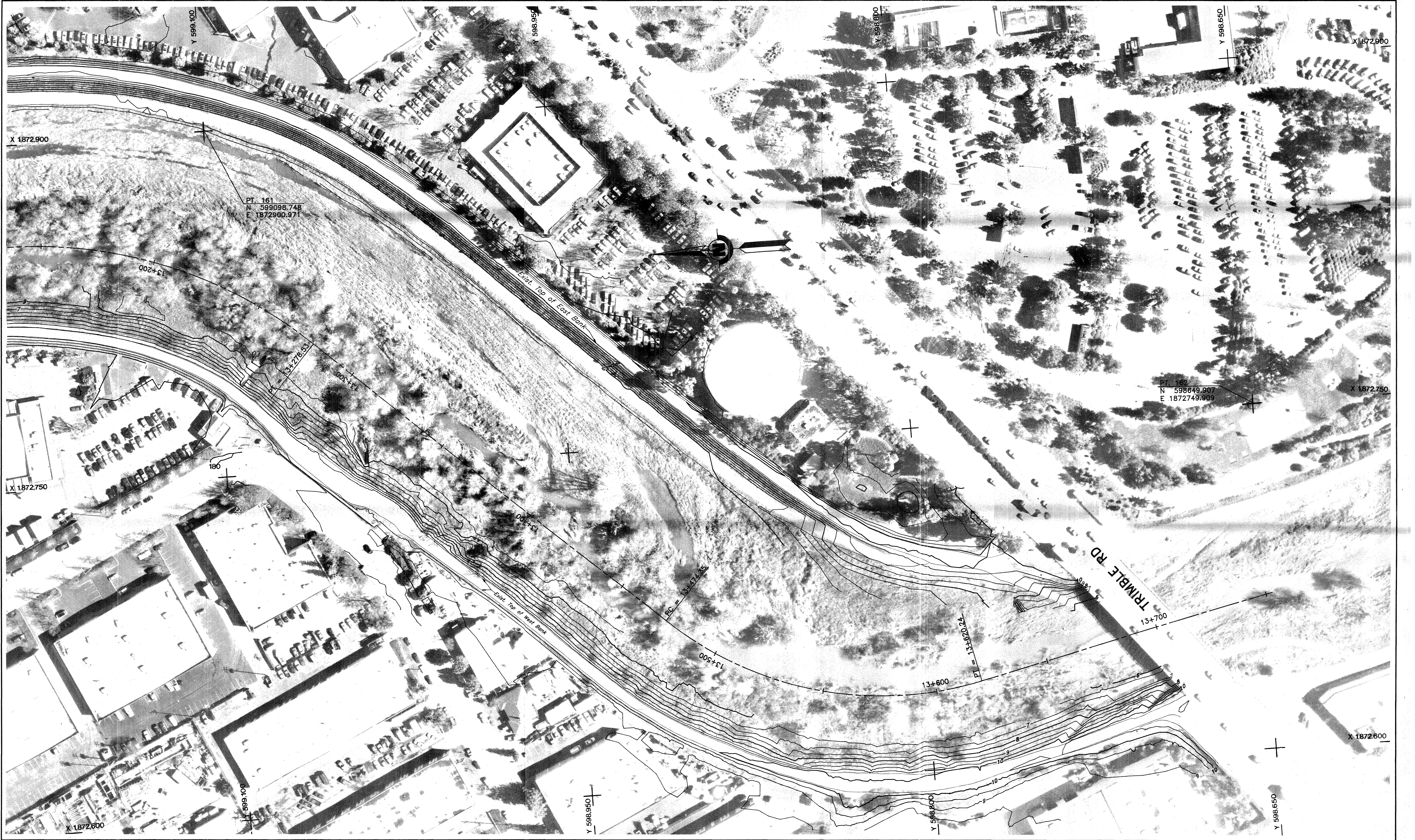
SCALE 1:750M

SHEET

Figure 3I

Aero-Geodetic Corporation 1-25-96 965 710

121



REV	DESCRIPTION	DATE	APPR	REFERENCE INFORMATION AND NOTES

DATE 11-25-97
DESIGN
DRAWN
CHECKED

Santa Clara Valley Water District

ENGINEERING CERTIFICATION

ENGINEERING APPROVAL

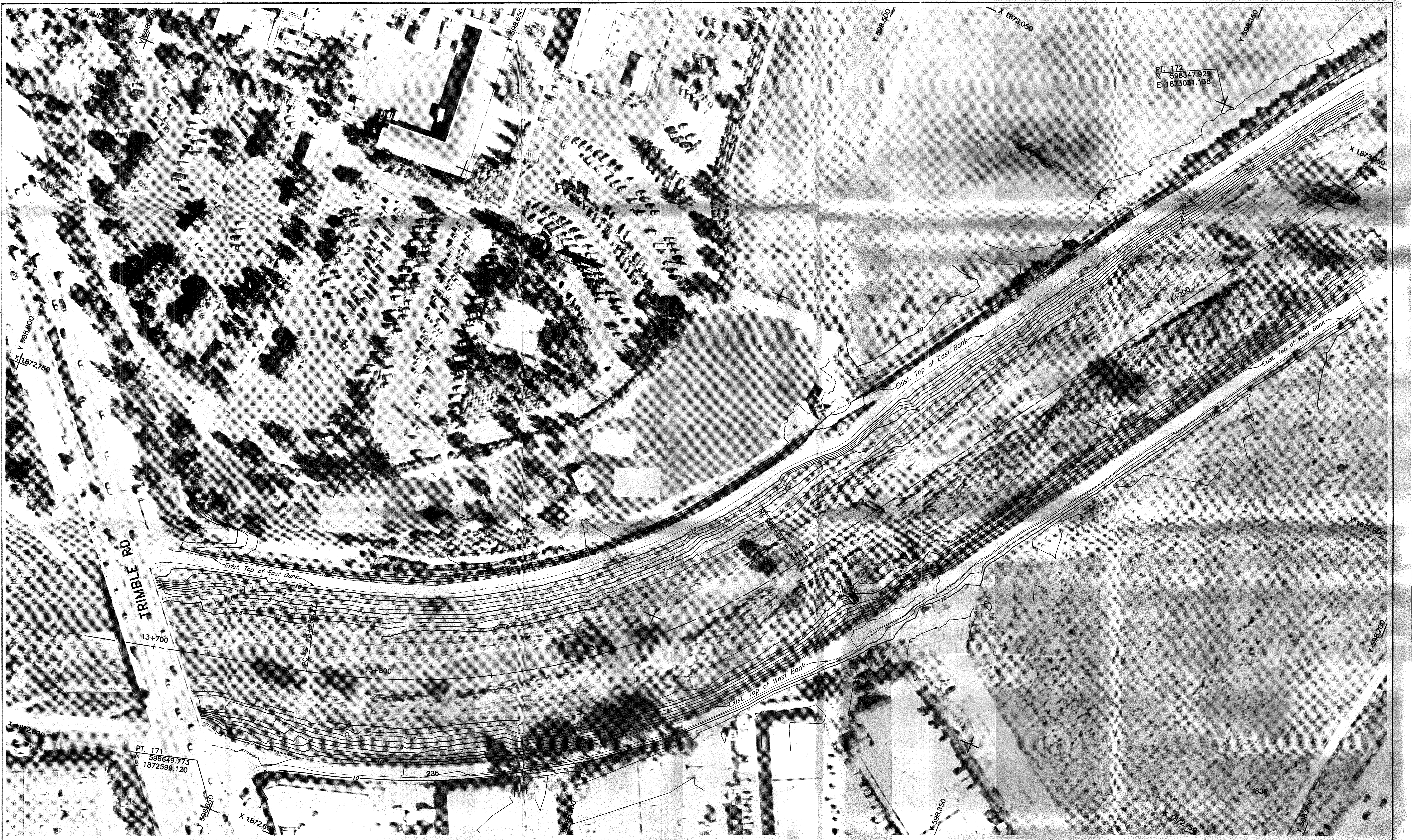
TOPOGRAPHIC MAP
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT
SANTA CLARA COUNTY, CALIFORNIA

SCALE 1 : 750 m

SHEET Figure 3m

Aero Geodetic Corp. [illegible] 965-1

13



REV	DESCRIPTION	DATE	APPR	REFERENCE INFORMATION AND NOTES

DATE
11-25-97
DESIGN
DRAWN
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Santa Clara Valley Water District

ENGINEERING CERTIFICATION

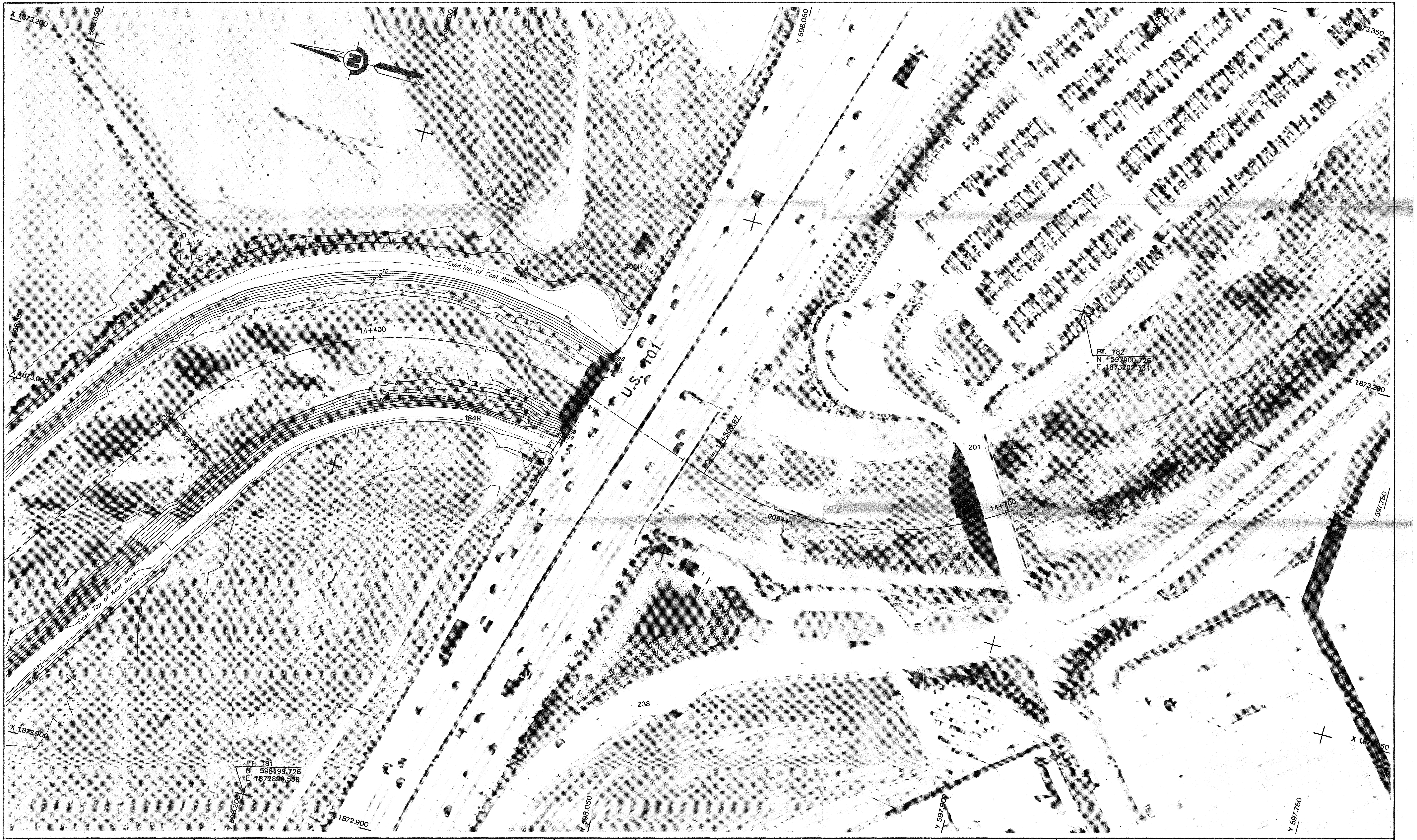
ENGINEERING APPROVAL

TOPOGRAPHIC MAP
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT
SANTA CLARA COUNTY, CALIFORNIA

SCALE
1 : 750 m

SHEET
Figure 3n

Aero Geodetic Corp [illegible] 96 965/[illegible]



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DATE 11-25-97
DESIGN
DRAWN
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Santa Clara Valley Water District

ENGINEERING CERTIFICATION

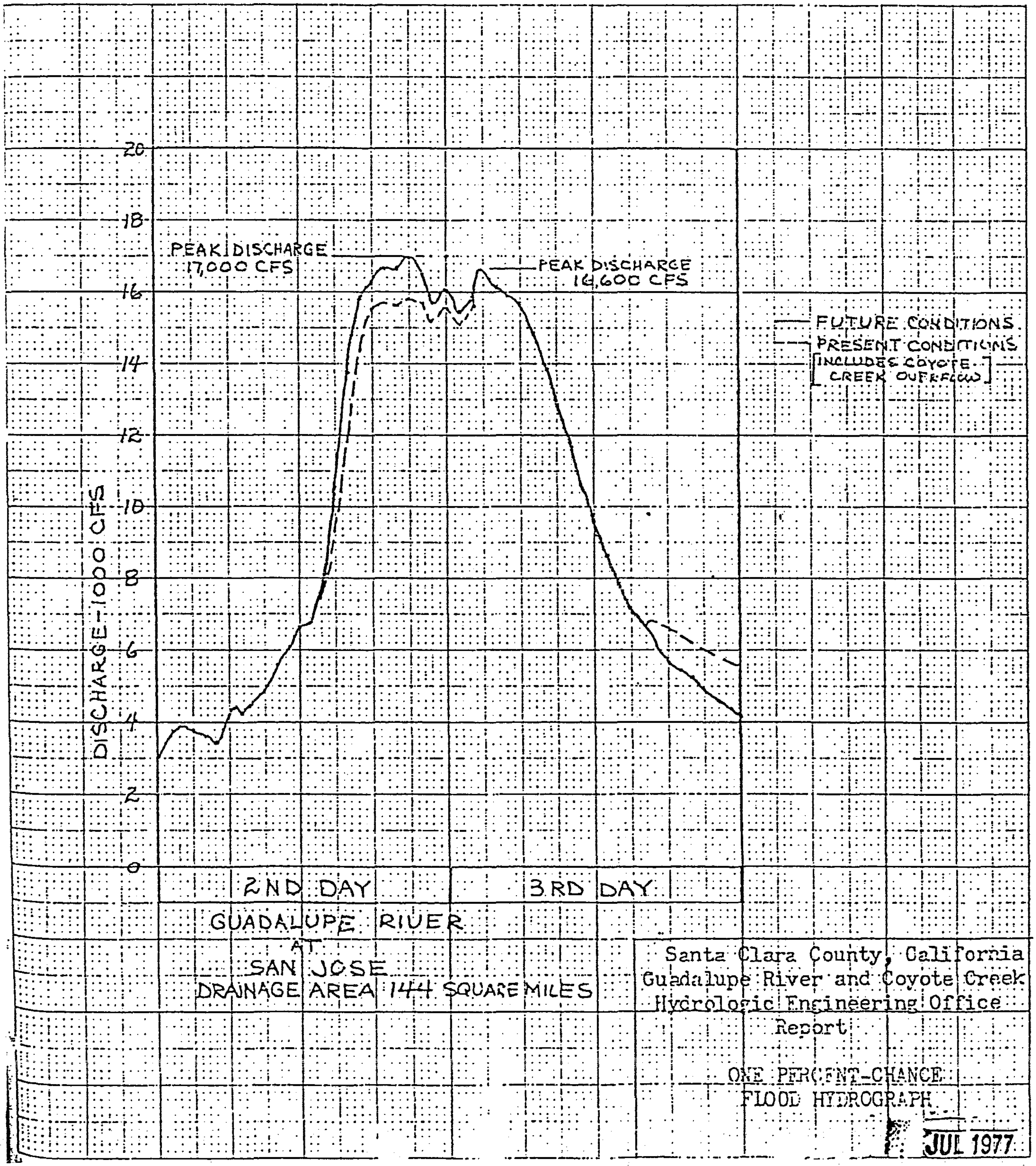
ENGINEERING APPROVAL

TOPOGRAPHIC MAP
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT
SANTA CLARA COUNTY, CALIFORNIA

SCALE
1 : 750 m

SHEET
Figure 3o

Aero Geodetic Corp. [illegible] 96 965716



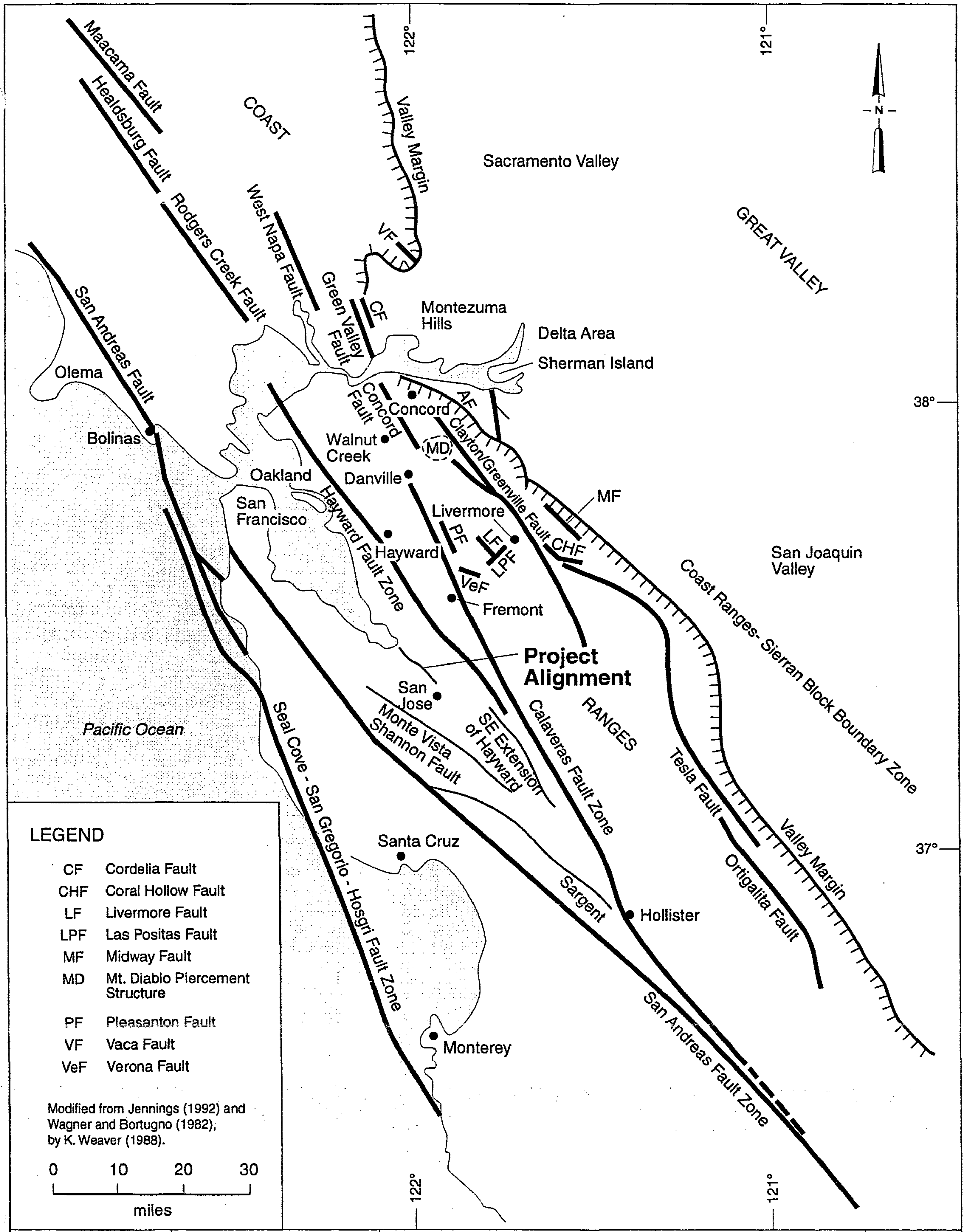
Santa Clara County, California
Guadalupe River and Coyote Creek
Hydrologic Engineering Office
Report

ONE PERCENT-CHANCE
FLOOD HYDROGRAPH

JUL 1977

Project No. 95-973017NB.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	ONE PERCENT CHANCE FLOOD HYDROGRAPH	Figure 4
06/30/1999			

URS Greiner Woodward Clyde



Modified from Jennings (1992) and Wagner and Bortugno (1982), by K. Weaver (1988).

Project No. 95-973017NB.00	Lower Guadalupe River Flood Control Project Santa Clara, California	REGIONAL ACTIVE FAULTS	Figure 5
URS Greiner Woodward Clyde			

CONTROL LINE STATION
EAST LEVEE REFERENCE LINE (METERS)
EAST LEVEE
LEGEND:
SILTY SAND (SM) CLAYEY SAND (SC), SILTY GRAVEL (GM), OR CLAYEY GRAVEL (GC)
WELL GRADED SAND (SW), OR GRAVEL (GW), OR POORLY-GRADED SAND (SP), OR GRAVEL (GP), OR DUAL SOIL TYPES: SW-SM, SW-SC, SP-SM, SP-SC, GW-GM, GW-GC, GP-GM OR GP-GC.
LEAN CLAY (CL) OR SILT (ML)
FAT CLAY (CH) ELASTIC SILT (MH), ORGANIC SILT OR CLAY (OL) OR (OH)
BAY MUD
FILL
Elevation (feet)
40
20
0
-20
-40
-60
10 TO 20 METERS OFFSET EAST LEVEE REFERENCE LINE
20 TO 40 METERS OFFSET EAST LEVEE REFERENCE LINE
40 TO 60 METERS OFFSET EAST LEVEE REFERENCE LINE
PYH-1
GRIN-8

NOTES:

1. THIS IDEALIZED SOIL PROFILE HAS BEEN CONSTRUCTED BY DIRECT INTERPOLATION BETWEEN BORINGS DRILLED AT VARYING SPACINGS AND PROJECTED TO THE PROFILE LINE. THE LINES CONNECTING THE VARIES LAYERS AT EACH BORING LOCATION ARE FOR SCHEMATIC ILLUSTRATION PURPOSES ONLY AND SHOULD NOT BE CONSTRUED TO REPRESENT THE ACTUAL CONDITIONS IN THE FIELD.
2. FOR DETAILED DESCRIPTIONS OF THE MATERIALS ENCOUNTERED IN EACH OF THE BORINGS DRILLED FOR THIS INVESTIGATION, SEE APPENDICES A AND E.
3. THE EXISTING GRADE INDICATED IS APPROXIMATE AND BASED UPON INTERPOLATIONS BETWEEN ELEVATION DATA EXTRACTED FROM CROSS-SECTIONS FURNISHED BY THE SANTA CLARA VALLEY WATER DISTRICT.

IDEALIZED SOIL PROFILE
FOR EAST LEVEE

Figure 6

Project No. 95-973017NB.00	Lower Guadalupe River Flood Control Project Santa Clara County, CA.
04/28/00	

CONTROL LINE STATION
WEST LEVEE REFERENCE LINE (METERS)
WEST LEVEE
Elevation (feet)
LEGEND:
SILTY SAND (SM) CLAYEY SAND (SC), SILTY GRAVEL (GM), OR CLAYEY GRAVEL (GC)
WELL GRADED SAND (SW), OR GRAVEL (GW), OR POORLY-GRADED SAND (SP), OR GRAVEL (GP), OR DUAL SOIL TYPES: SW-SM, SW-SC, SP-SM, SP-SC, GW-GM, GW-GC, GP-GM OR GP-GC.
LEAN CLAY (CL) OR SILT (ML)
FAT CLAY (CH) ELASTIC SILT (MH), ORGANIC SILT OR CLAY (OL) OR (OH)
BAY MUD
FILL
10 TO 20 METERS OFFSET WEST LEVEE REFERENCE LINE
20 TO 40 METERS OFFSET WEST LEVEE REFERENCE LINE
40 TO 60 METERS OFFSET WEST LEVEE REFERENCE LINE

NOTES:

1. THIS IDEALIZED SOIL PROFILE HAS BEEN CONSTRUCTED BY DIRECT INTERPOLATION BETWEEN BORINGS DRILLED AT VARYING SPACINGS AND PROJECTED TO THE PROFILE LINE. THE LINES CONNECTING THE VARIES LAYERS AT EACH BORING LOCATION ARE FOR SCHEMATIC ILLUSTRATION PURPOSES ONLY AND SHOULD NOT BE CONSTRUED TO REPRESENT THE ACTUAL CONDITIONS IN THE FIELD.
2. FOR DETAILED DESCRIPTIONS OF THE MATERIALS ENCOUNTERED IN EACH OF THE BORINGS DRILLED FOR THIS INVESTIGATION, SEE APPENDICES A AND E.
3. THE EXISTING GRADE INDICATED IS APPROXIMATE AND BASED UPON INTERPOLATIONS BETWEEN ELEVATION DATA EXTRACTED FROM CROSS-SECTIONS FURNISHED BY THE SANTA CLARA VALLEY WATER DISTRICT.

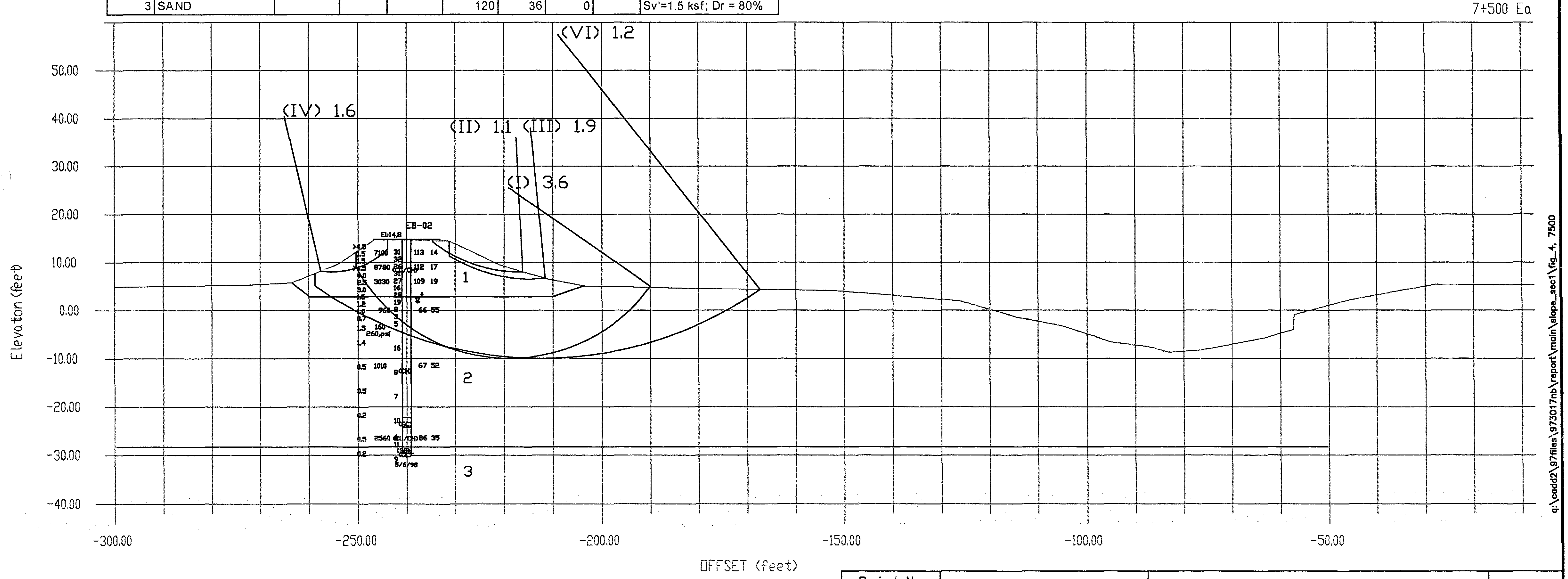
IDEALIZED SOIL PROFILE
FOR WEST LEVEE

Figure 7

Project No. 95-973017NB.00	Lower Guadalupe River Flood Control Project Santa Clara County, CA.
04/28/00	

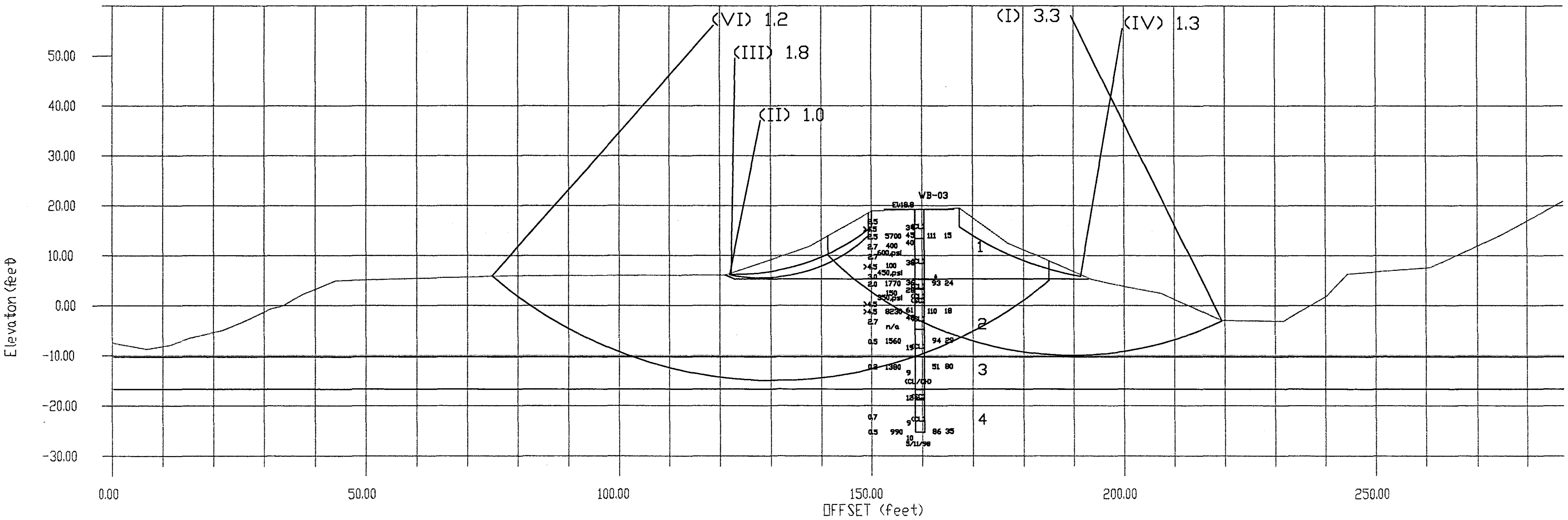
Summary of Geotechnical Design Parameters

Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	φ' (deg)	c' (psf)	Su (psf)	
1		Table 3				33	0		
		EB-01-6				27	230		
	Lean to Fat CLAY (FILL)		110	18	130	33	0	2500	Selected
2		Table 3				20	110		
		GRIV-19-2				28	0		
	Fat CLAY		60	66	100	20	110	600	Selected
3	SAND				120	36	0		Estimated from N=25, Sv'=1.5 ksf; Dr = 80%



Project No. 95-973017NB.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT San Clara County, California	STABILITY ANALYSIS CROSS-SECTION 1 STATION 7+500 EAST LEVEE	Figure 8
TEMP004 07/01/99			
URS Greiner Woodward Clyde			

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	φ' (deg)	c' (psf)	Su (psf)	
1		Table 3				33	0		
		WB-03-6				31	288		
	Lean CLAY (FILL)		110	18	130	33	0	4000	Selected
2		Table 3				28	100		
		WB-03-12				27	590		
	Lean CLAY		100	20	120	28	100	1000	Selected
3		Table 3				20	110		
		GRIV-16-2				26.5	0		
	Fat CLAY		60	66	100	20	110	750	Selected
4	Lean CLAY		90	30	117	28	100	500	Selected



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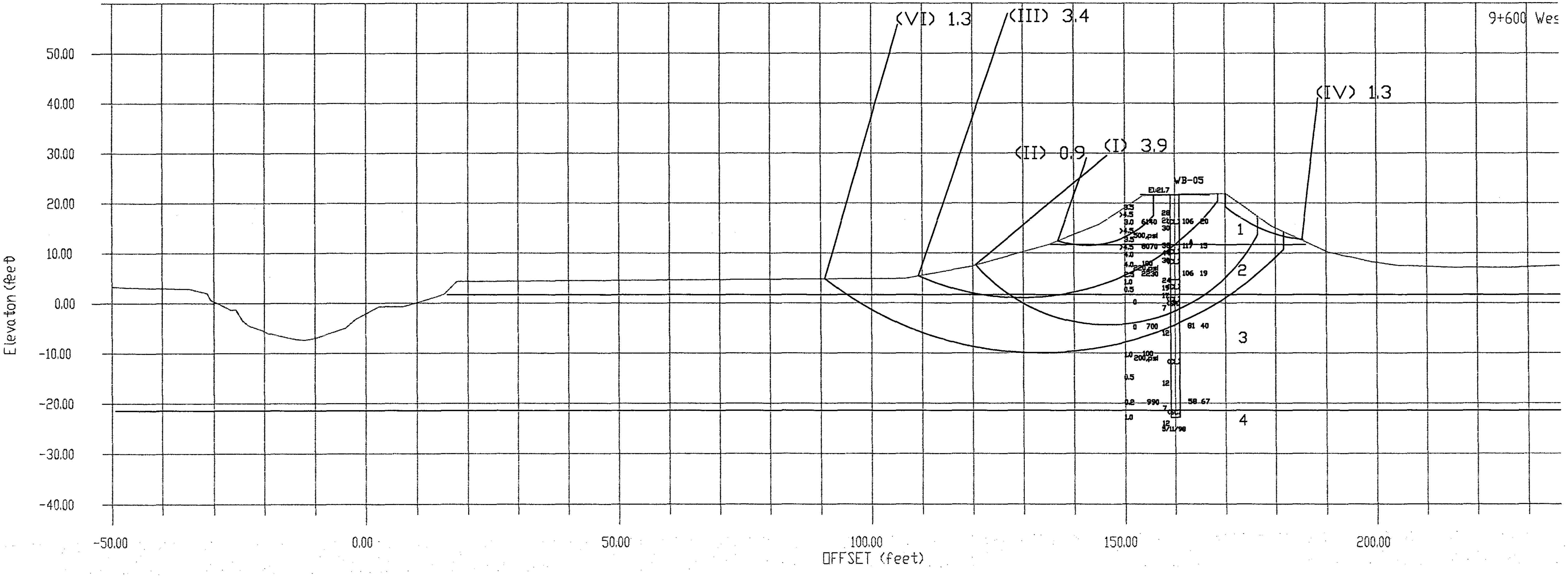
Project No. 95-973017NB.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT San Clara County, California	STABILITY ANALYSIS CROSS-SECTION 2 STATION 8+610 WEST LEVEE	Figure 9
TEMP004 07/01/99			
URS Greiner Woodward Clyde			

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	φ' (deg)	c' (psf)	Su (psf)	
1		Table 3				33	0		
		WB-05-4				29	290		
	Lean CLAY (FILL)		110	18	130	33	0	4000	Selected
2		Table 3				28	100		
		GRIV-7-2				35	0		
	Lean CLAY		97	24	120	28	100	1000	Selected
3		Table 3				20	110		
		WB-05-14				20*	0*		UU Total Stress
	Fat CLAY		80	40	112	20	110	900	Selected
4	Clayey SAND				125	36	0		Estimated from N = 16, Sv' = 3.5 ksf; Dr = 55 %



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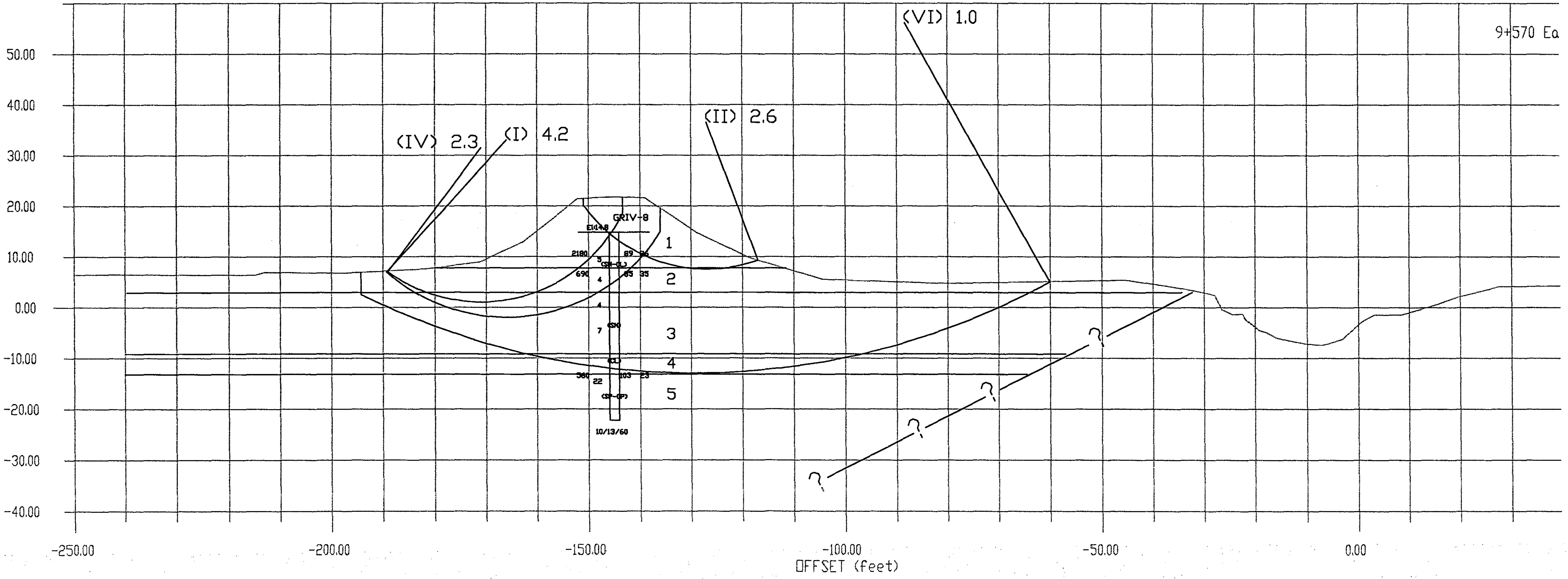
Project No. 95-973017NB.00 07/01/99	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT San Clara County, California	STABILITY ANALYSIS CROSS-SECTION 3 STATION 9+600 WEST LEVEE	Figure 10
URS Greiner Woodward Clyde			

Summary of Geotechnical Design Parameters

Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	
1		Table 3				24	450		
		EB-09				23	750		
	Lean CLAY (FILL)		108	20	130	24	450	2700	Selected
2		Table 3				28	100		
		EB-09				18	640		
	Lean CLAY		100	20	120	28	100	1000	Selected
3	Silty SAND				120	33	0		Estimated from N = 5, Sv' = 2.6 ksf; Dr = 25 %
4		Table 3				30	175		
	Lean CLAY		103	23	127	30	175	300	Selected
5	SAND				125	36	0		Estimated from N = 22, Sv' = 3.7 ksf; Dr = 65 %



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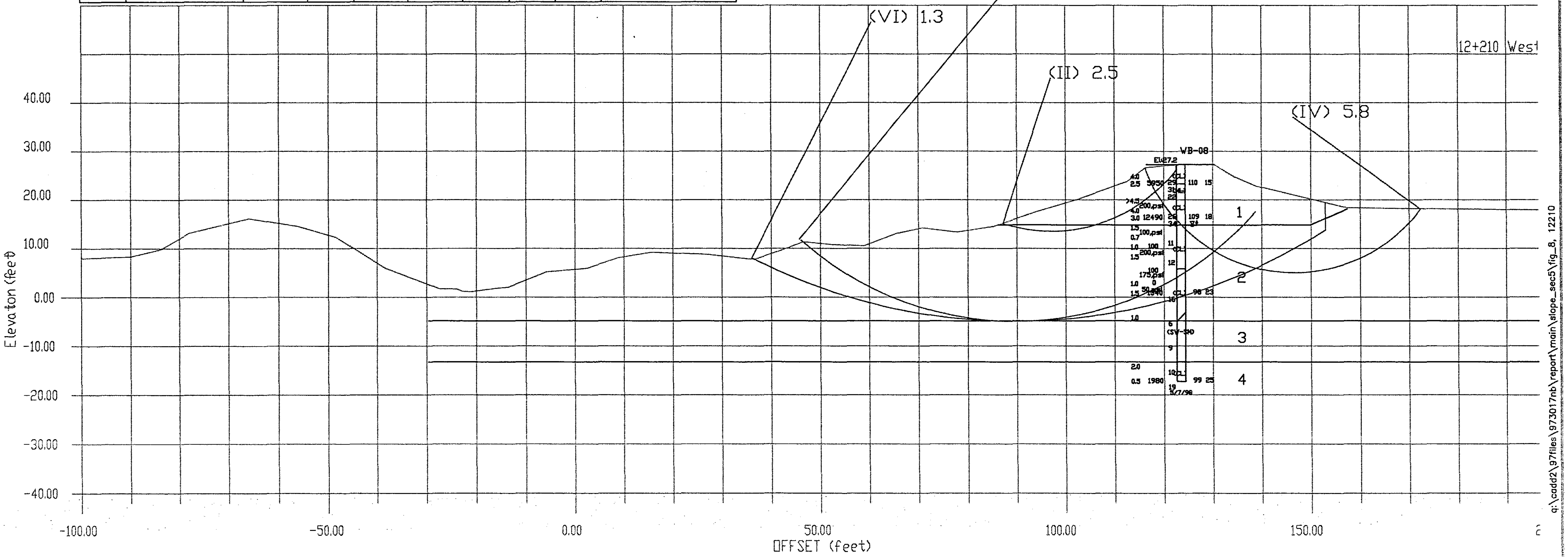
Project No. 95-973017NB.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT San Clara County, California	STABILITY ANALYSIS CROSS-SECTION 4 STATION 9+570 EAST LEVEE	Figure 11
07/01/99			
URS Greiner Woodward Clyde			

Summary of Geotechnical Design Parameters

Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	
1		Table 3				24	450		
		WB-08-7				18.5	475		
	Lean CLAY (FILL)		110	14	125	24	450	4000	Selected
2		Table 3				28	100		
		WB-08-12				25	170		
	Lean CLAY		90	30	117	28	100	1000	Selected
3	SAND				115	34	0		Estimated from N = 8, Sv' = 2.6 ksf; Dr = 40 %
4		Table 3				28	100		
	Lean CLAY		100	25	125	28	100	1000	Selected



Project No.
95-973017NB.00
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LOWER GUADALUPE RIVER
FLOOD CONTROL PROJECT
San Clara County, California

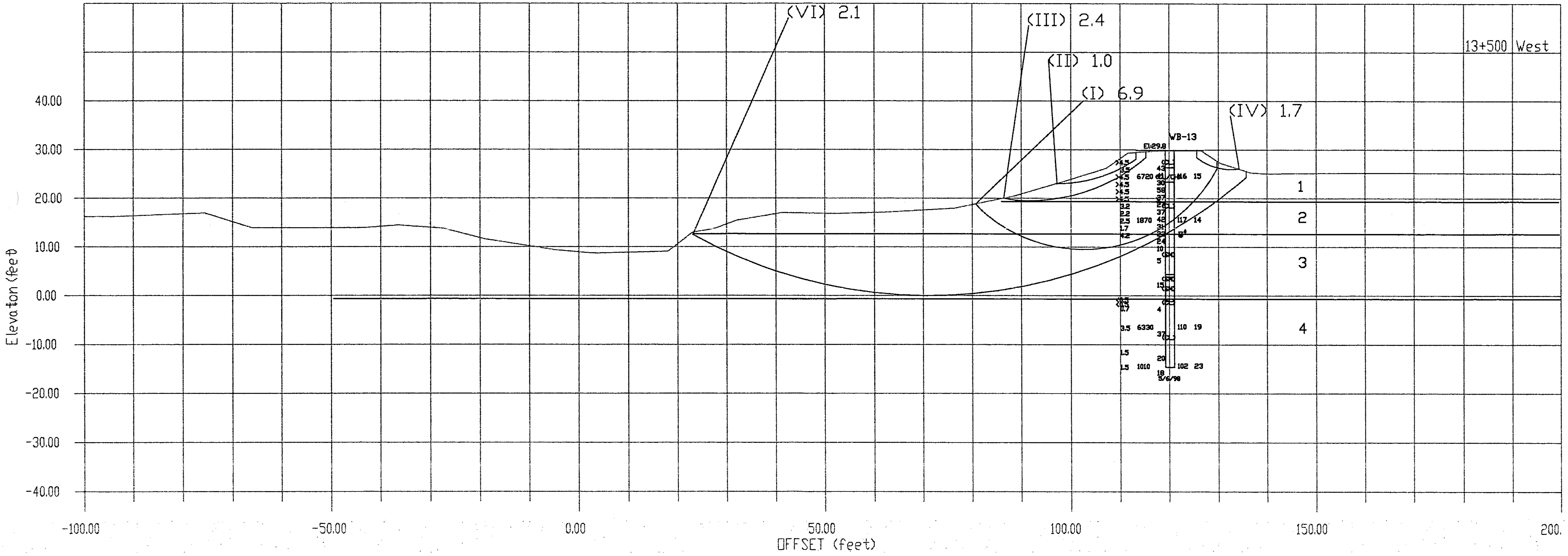
URS Greiner Woodward Clyde

STABILITY ANALYSIS
CROSS-SECTION 5
STATION 12+210 WEST LEVEE

Figure
12

Summary of Geotechnical Design Parameters

Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	φ' (deg)	c' (psf)	Su (psf)	
1	Lean to Fat CLAY (FILL)	Table 3	115	15	132	33	0	3000	Selected
2		Table 3				24	450		
		WB-8				25	170		
	Lean CLAY (FILL)		108	20	130	24	450	1400	Selected
3	Silty SAND				125	35	0		Estimated from N = 10, Sv' = 2.7 ksf; Dr = 50 %
4	Lean CLAY	Table 3				28	100		
		WB-8				25	170		
			100	25	125	28	100	1700	Selected



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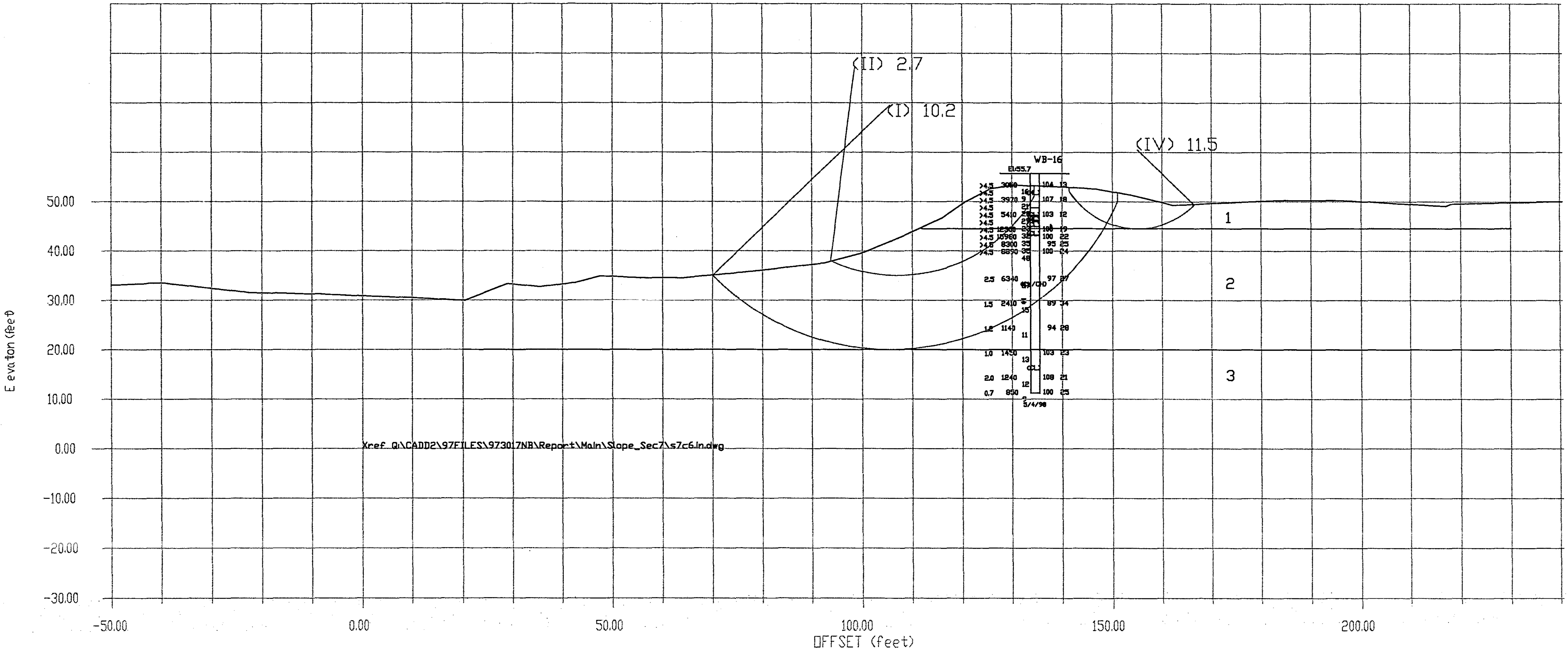
Project No. 95-973017NB.00 TEMP004 07/01/99	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT San Clara County, California	STABILITY ANALYSIS CROSS-SECTION 6 STATION 13+500 WEST LEVEE	Figure 13
URS Greiner Woodward Clyde			

Summary of Geotechnical Design Parameters

Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	
1	Lean CLAY (FILL)	Table 3	100	20	120	24	450	3500	Selected
2		Table 3				28	100		
		WB-15				27	590		
	Fat to Lean CLAY		95	25	119	28	100	1500	Selected
3		Table 3				28	100		
		WB-17				24	620		
	Lean CLAY		100	25	125	28	100	900	Selected

16+620 We



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Project No. 95-973017NB.00 07/01/99	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT San Clara County, California	STABILITY ANALYSIS CROSS-SECTION 7 STATION 16+620 WEST LEVEE	Figure 14

TEMP004

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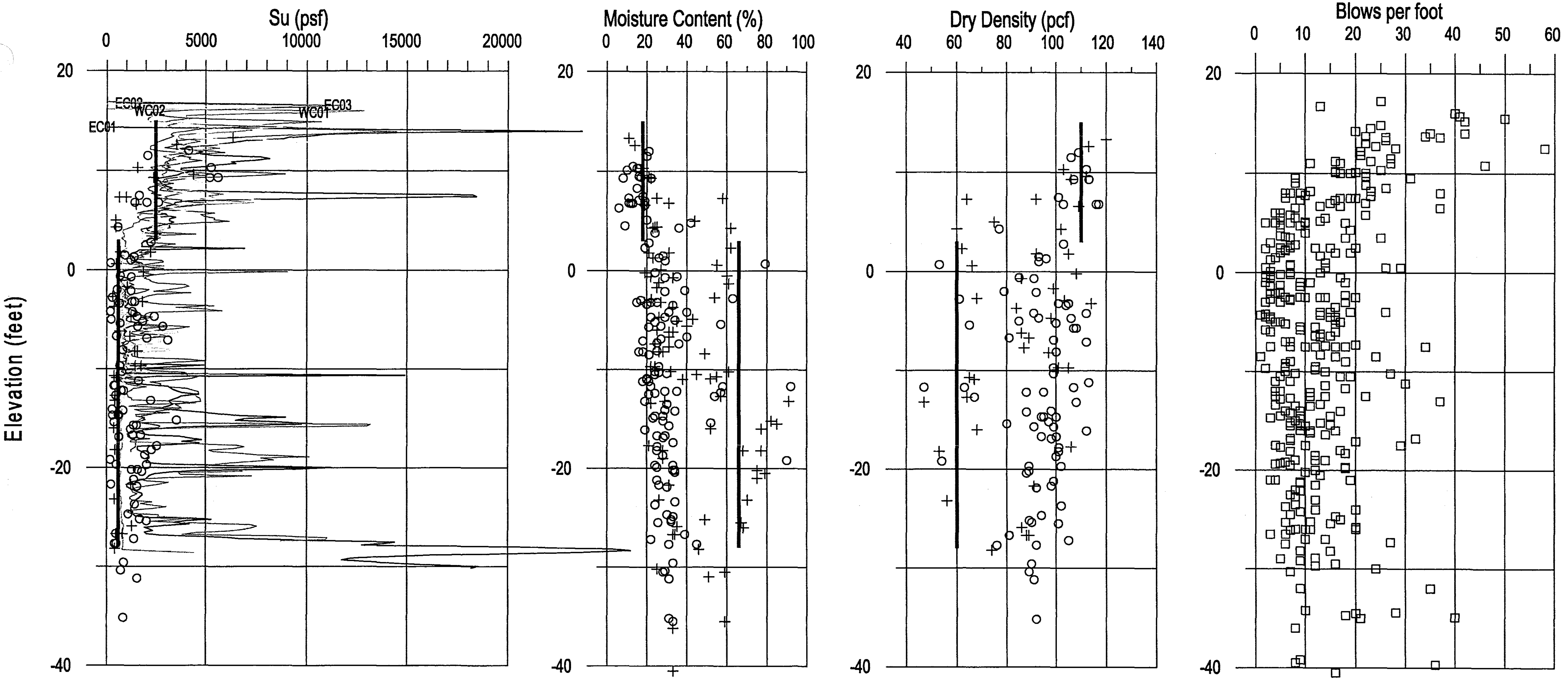


Data from the following borings and CPTs:

BSP-1
BSP-2
BSP-3
GJJ-6
GRI-EB1
GRI-EB10
GRI-EB2
GRI-EB3
GRI-EB4
GRI-EB5
GRI-EB6
GRI-EB7
GRI-EB8
GRI-EB9
GRIM-6
GRIM-7
GRIV-17
GRIV-18
GRIV-19
GRIV-20
GRIV-21
GROCS-1
GSB-1
GSB-2
GSB-2A
GSB-2B
GSB-3
GSB-4
GSB-5
GSBOLD-1
GSBOLD-2
GSBOLD-3
GSBOLD-4
GSBOLD-5
GSBOLD-6
RREMB-1
RREMB-2
RREMB-3
RREMB-4
RREMB-5
RREMB-6
SPRRB-R1
SPRRB-R2
SPRRB-R3
SPRRB-R4
SPRRB-R5
SPRRB-R6
SPRRB-R7
EB-02
EB-03
EB-04
EB-05
EB-06
EB-07
WB-01
WB-02

EC-01 EC-02 EC-03
WC-01 WC-02

+ Fat Clay

○ Lean Clay

□ All Soil Types

| Selected Design Parameter for Depth Intervals Shown

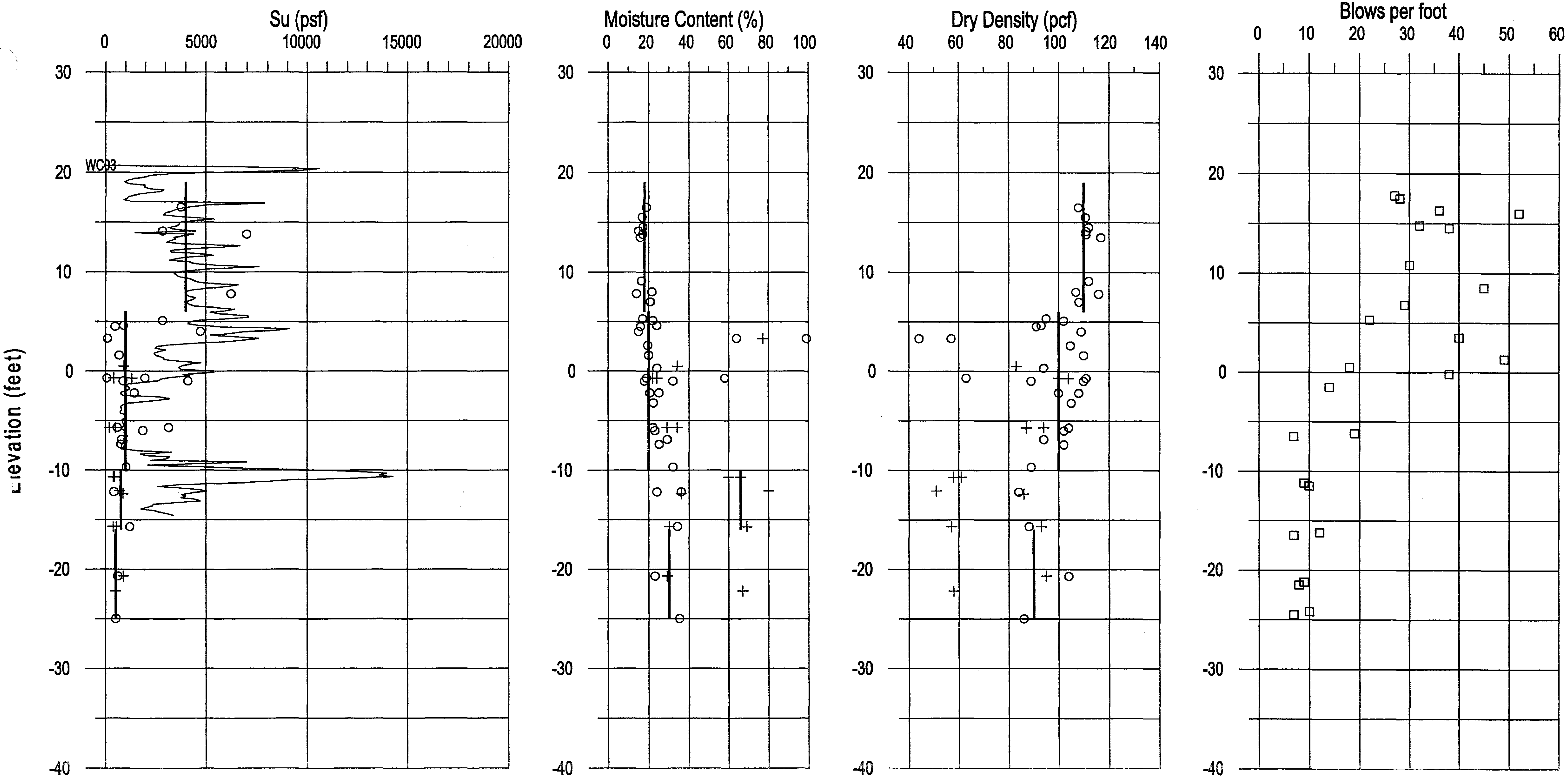
Project No.
95-9730717NB.00
06/30/1999

LOWER GUADALUPE RIVER
FLOOD CONTROL PROJECT
Santa Clara, California

URS Greiner Woodward Clyde

INDEX PROPERTY PLOTS FOR
CROSS-SECTION No. 1

Figure
15



Data from the following borings and CPT:

GRIV-15
GRIV-16
EB-08
WB-03

WC-03

○ Lean Clay

+ Fat Clay

□ All Soil Types

| Selected Design Parameter for Depth Intervals Shown

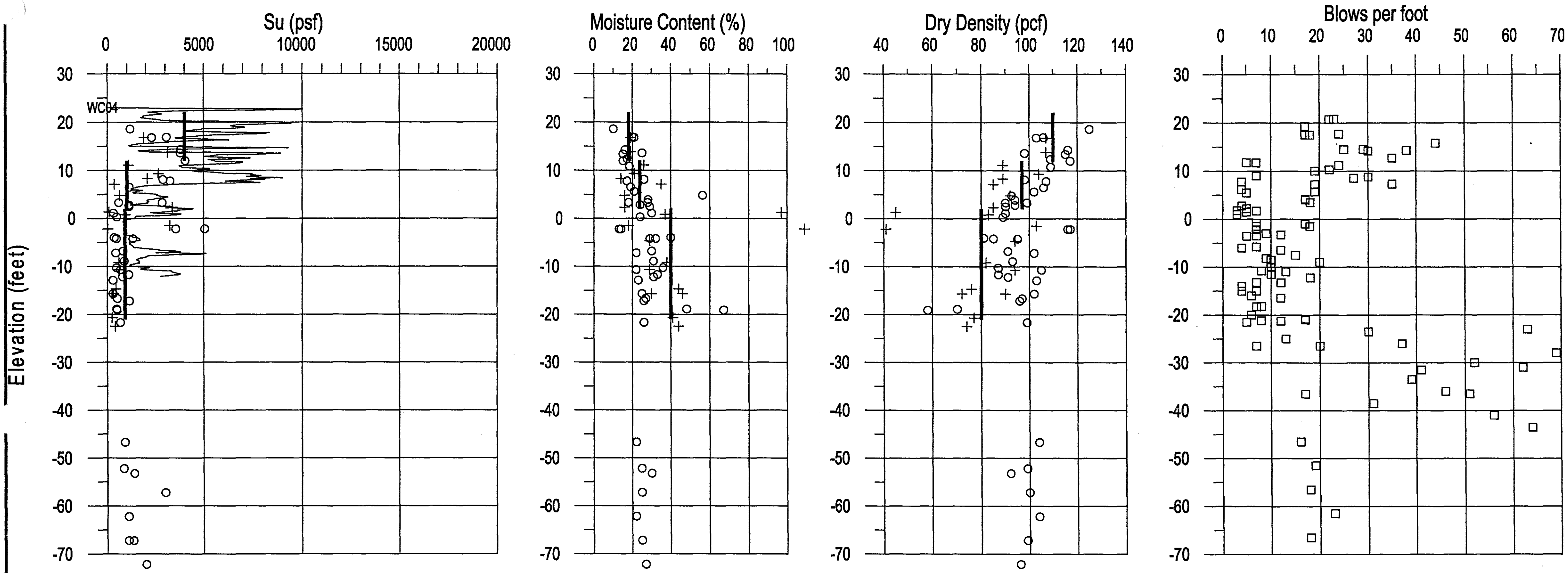
Project No.
95-9730717NB.00
06/30/1999

LOWER GUADALUPE RIVER
FLOOD CONTROL PROJECT
Santa Clara, California

URS Greiner Woodward Clyde

INDEX PROPERTY PLOTS FOR
CROSS-SECTION No. 2

Figure
16



Data from the following borings and CPT:

GRIV-10
GRIV-7
GRIV-8
GRIV-9
TDB-1
TDB-2
TDB-3
TDB-4
TDB-5
EB-09
WB-05

WC-04

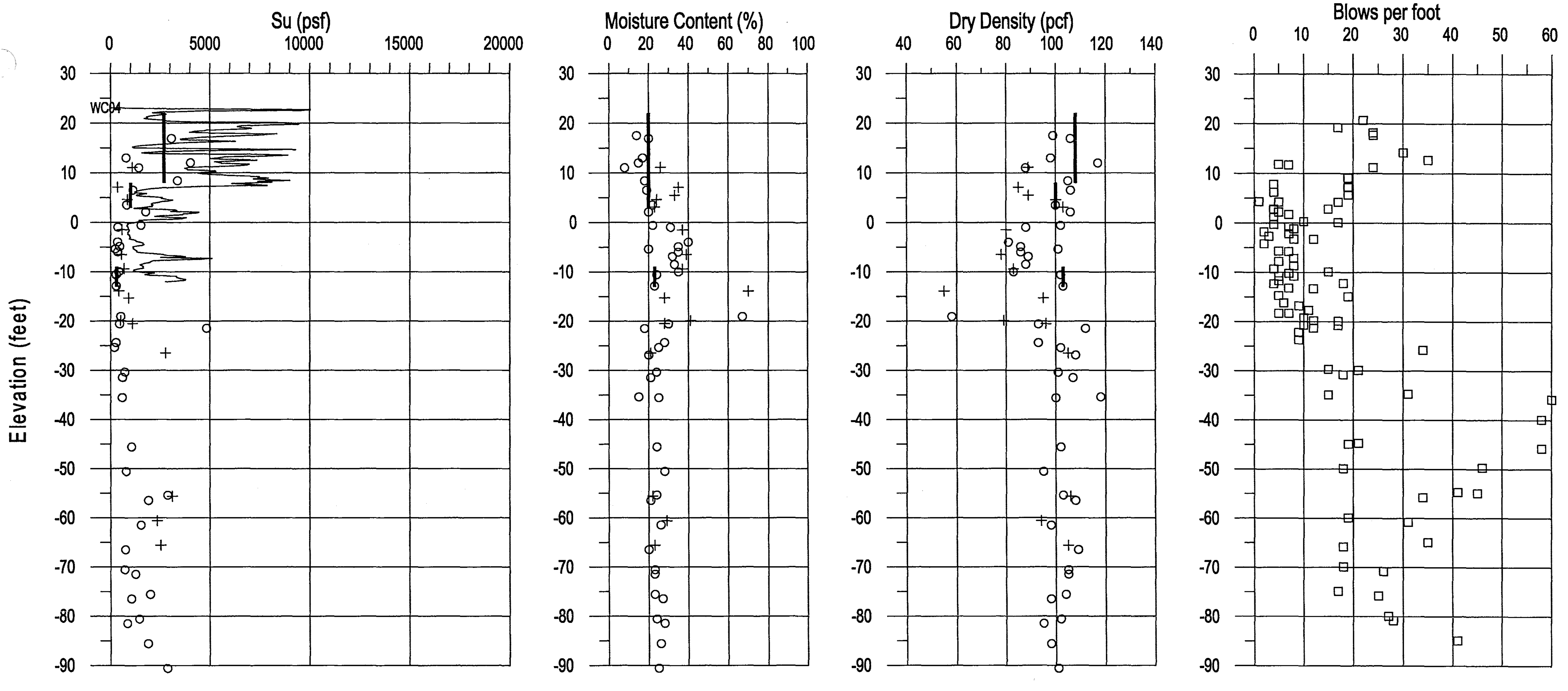
+ Fat Clay

○ Lean Clay

□ All Soil Types

| **Selected Design Parameter for Depth Intervals Shown**

Project No. 95-9730717NB.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	INDEX PROPERTY PLOTS FOR CROSS-SECTION No. 3	Figure 17
06/30/1999			
URS Greiner Woodward Clyde			



```
Data from the following borigs CPT:

GRIV-7
GRIV-8
HHP-1
HHP-3
HHP-4
WB-05

WC-04
```

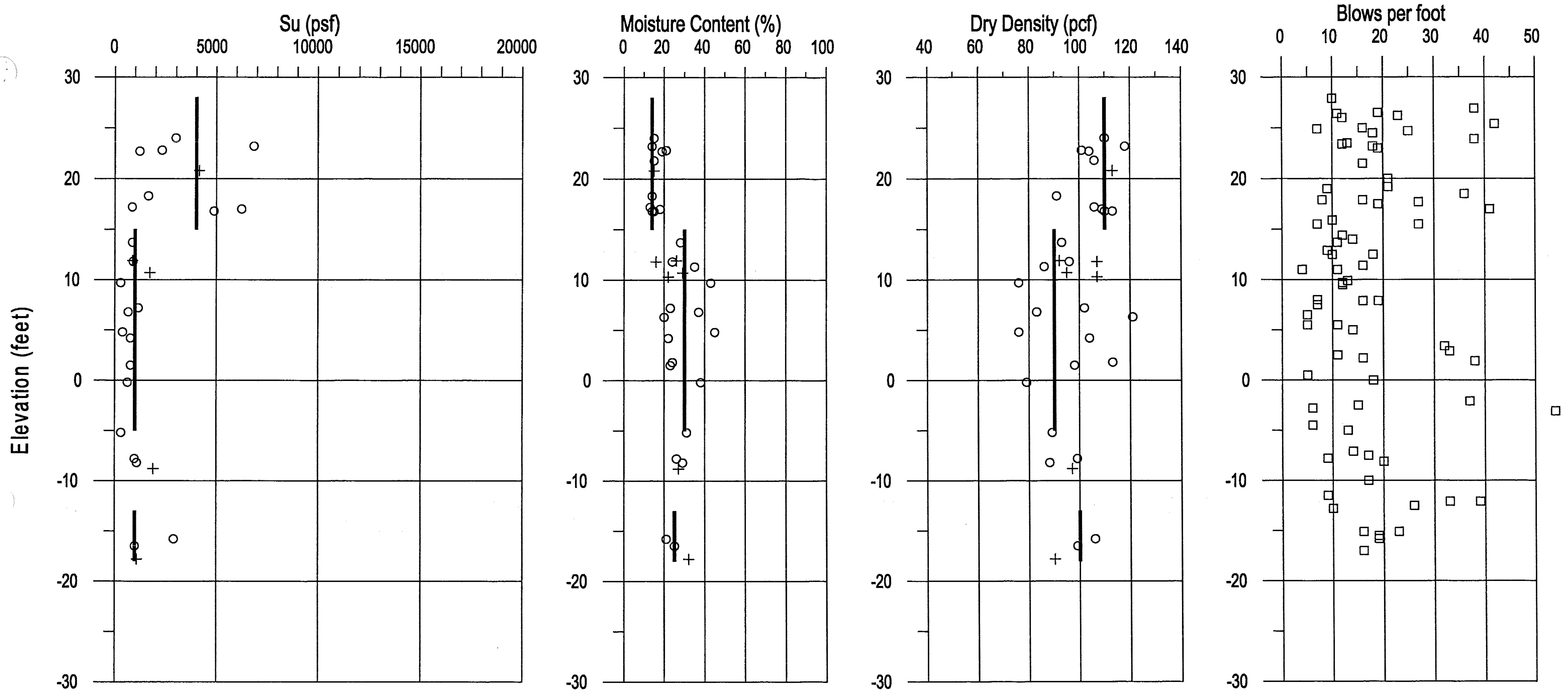
+ Fat Clay

○ Lean Clay

☐ All Soil Types

| Selected Design Parameter for Depth Intervals Shown

Project No. 95-9730717NB.00 06/30/1999	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	INDEX PROPERTY PLOTS FOR CROSS-SECTION No. 4	Figure 18
URS Greiner Woodward Clyde			



Data from the following borings:

GRIV-1
SPRR-1
SPRR-2
SPRR-3
SPRR-4
SPRR-7
SPRR-8
SPRR-9
EB-13
WB-07
WB-08
WB-09
WB-10

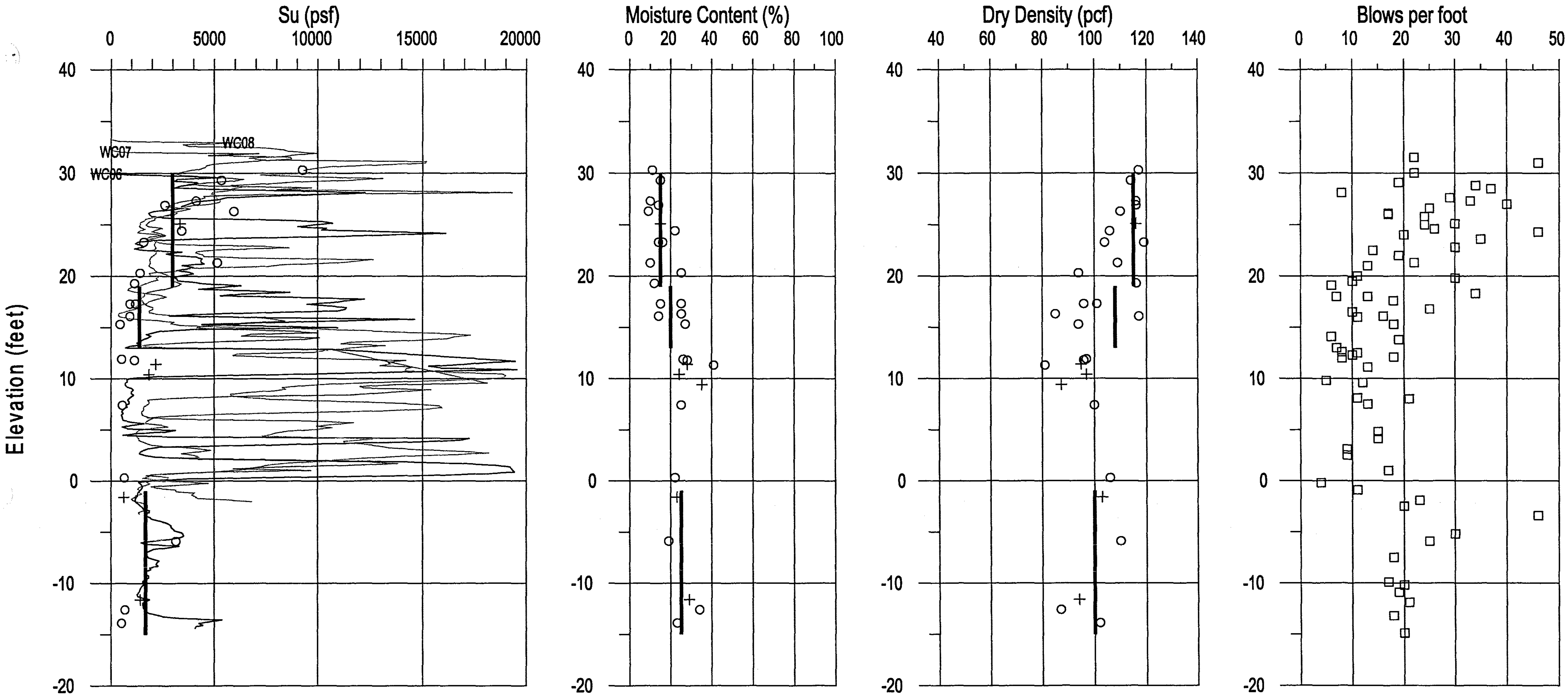
+ Fat Clay

○ Lean Clay

☐ All Soil Types

| Selected Design Parameter for Depth Intervals Shown

Project No. 95-9730717NB.00 06/30/1999	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	INDEX PROPERTY PLOTS FOR CROSS-SECTION No. 5
URS Greiner Woodward Clyde		Figure 19



Data from the following borings and CPTs:

SJIA-S11
SJIA-S12
SPRR-10
SPRR-5
SPRR-6
EB-14
EB-15
WB-12
WB-13

WC-06
WC-07
WC-08

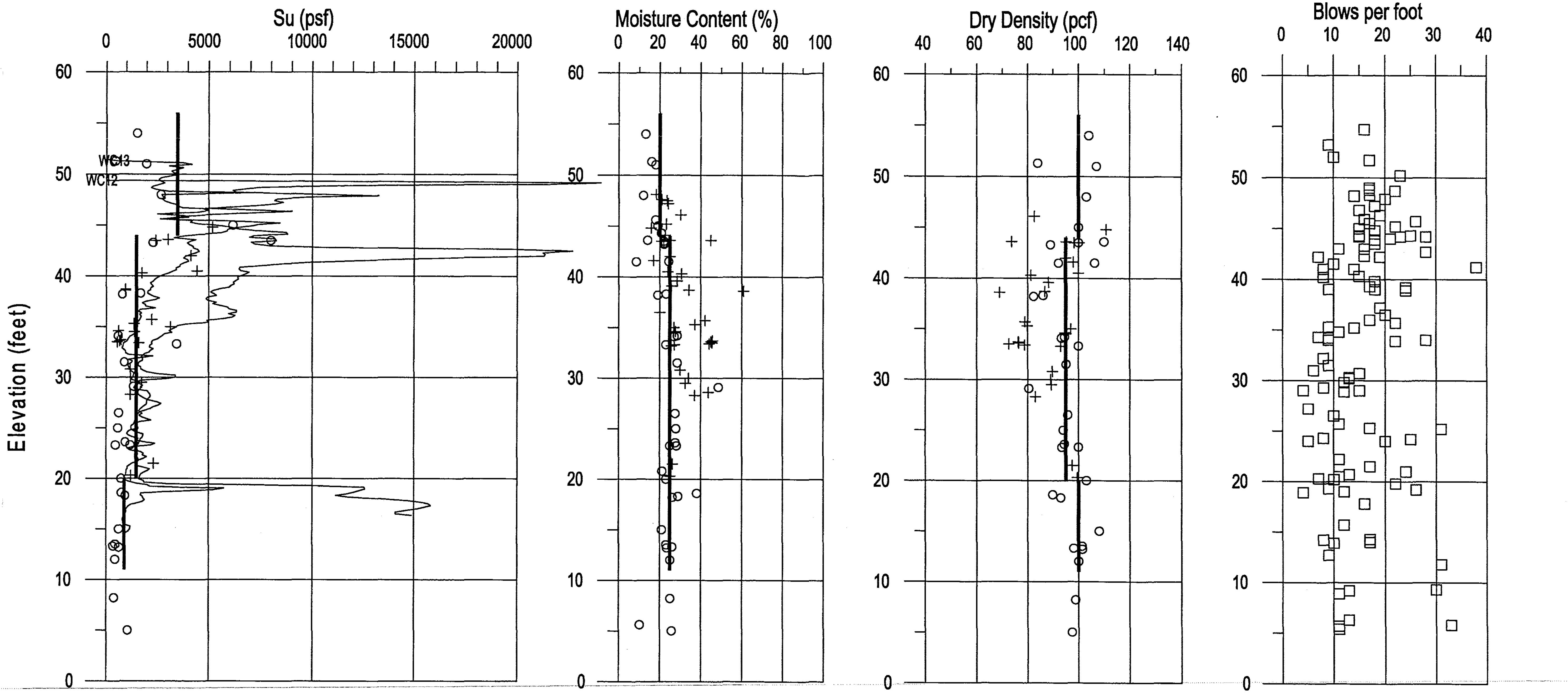
+ Fat Clay

○ Lean Clay

□ All Soil Types

| Selected Design Parameter for Depth Intervals Shown

Project No. 95-9730717NB.00 06/30/1999	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	INDEX PROPERTY PLOTS FOR CROSS-SECTION No. 6	Figure 20
URS Greiner Woodward Clyde			



Data from the following borings:

RHM-R9
SDB-10
SDB-11
SDB-12
SDB-13
SDB-14
SDB-15
SDB-16
SDB-7
SDB-8
SDB-9
WB-16

WC-12
WC-13

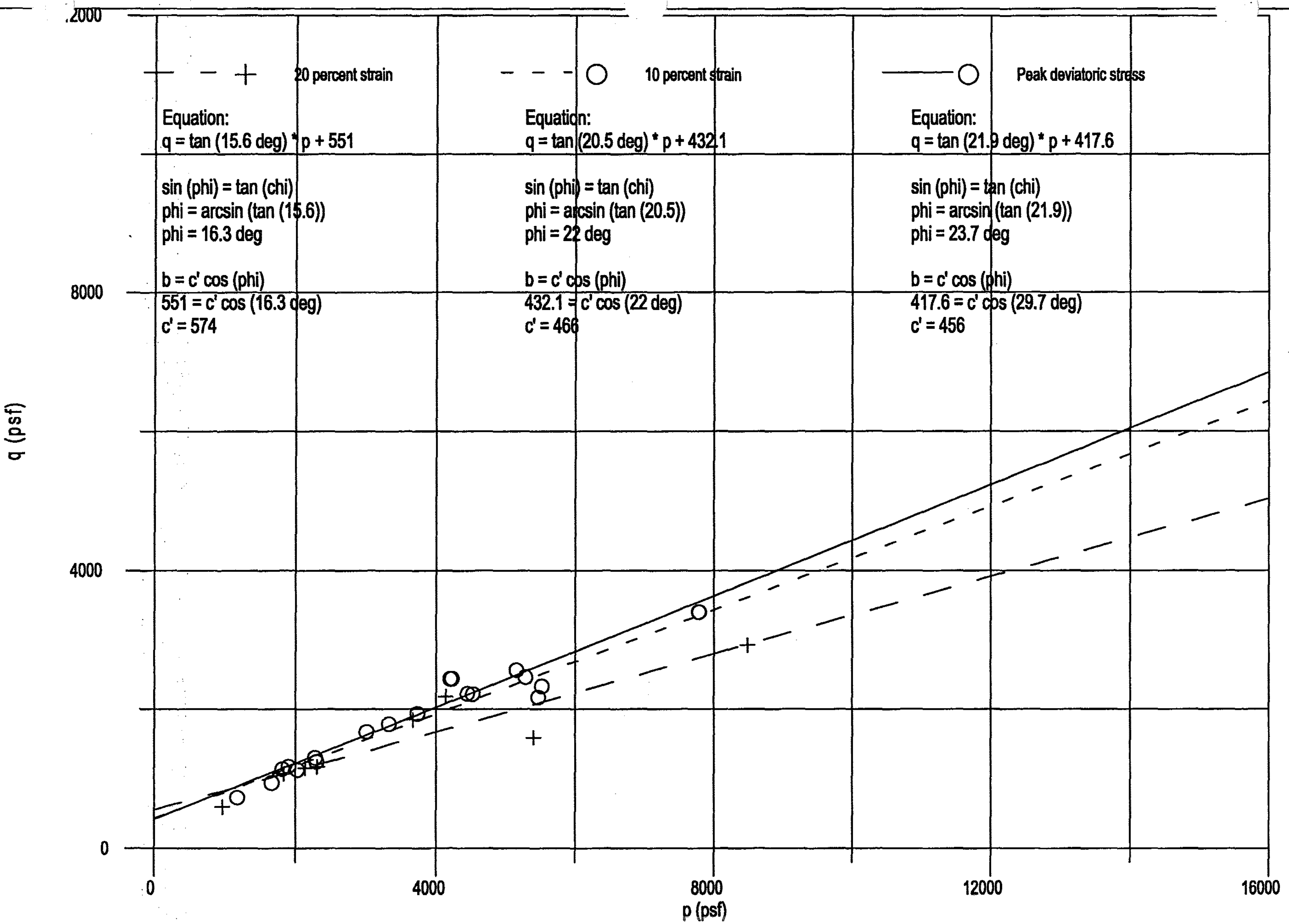
+ Fat Clay

○ Lean Clay

□ All soil types

| **Selected Design Parameter for Depth Intervals Shown**

Project No. 95-9730717NB.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	INDEX PROPERTY PLOTS FOR CROSS-SECTION No. 7	Figure 21
06/30/1999			
URS Greiner Woodward Clyde			



Project No.
95-973017NB.00

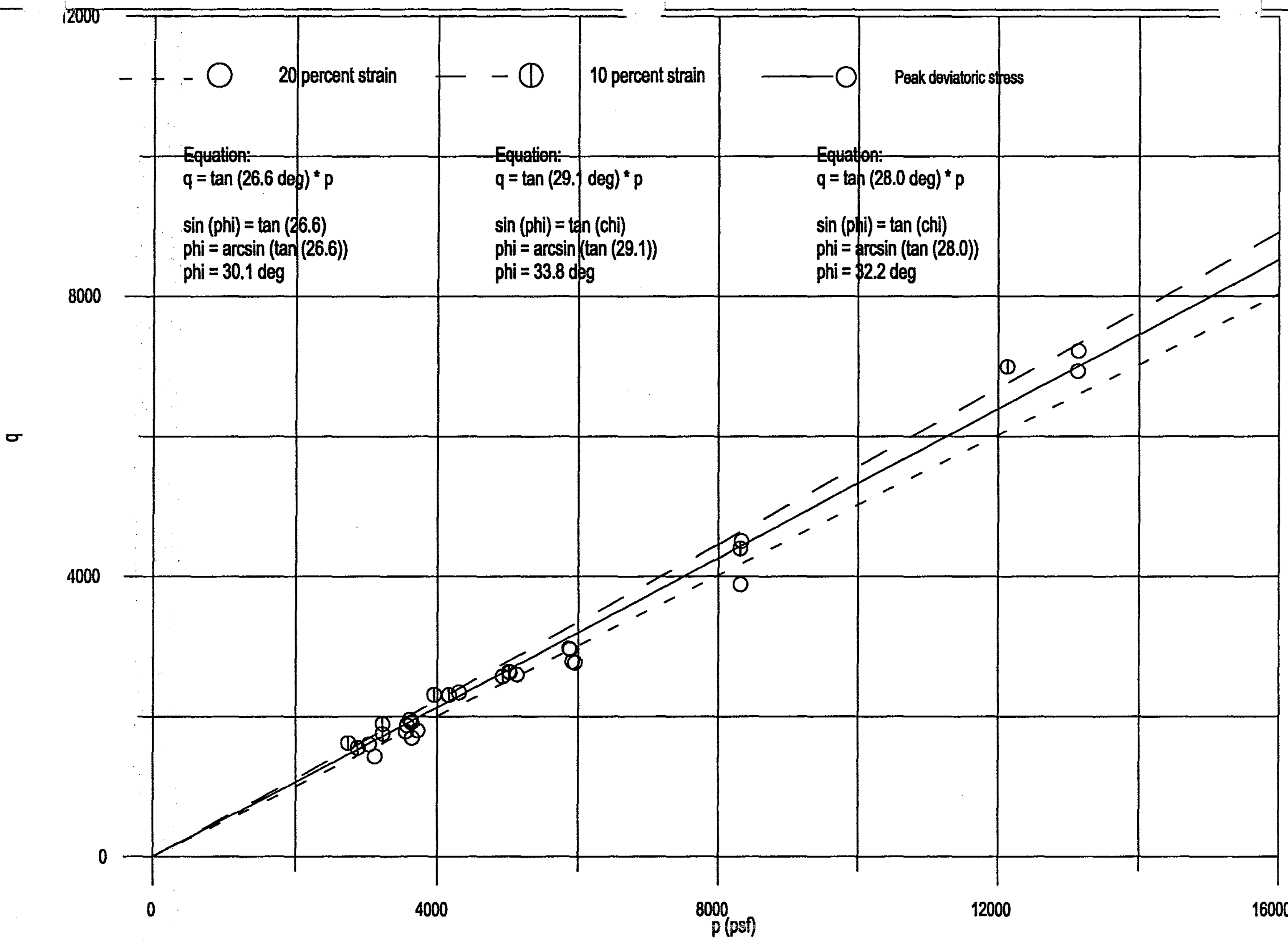
07/02/1999

LOWER GUADALUPE RIVER
FLOOD CONTROL PROJECT
Santa Clara, California

URS Greiner Woodward Clyde

Shear Strength Parameters
For
Lean Clay Fill with Dry Density less than 110 pcf

Figure
22



Equation:
q = tan (26.6 deg) * p

sin (phi) = tan (26.6)
phi = arcsin (tan (26.6))
phi = 30.1 deg

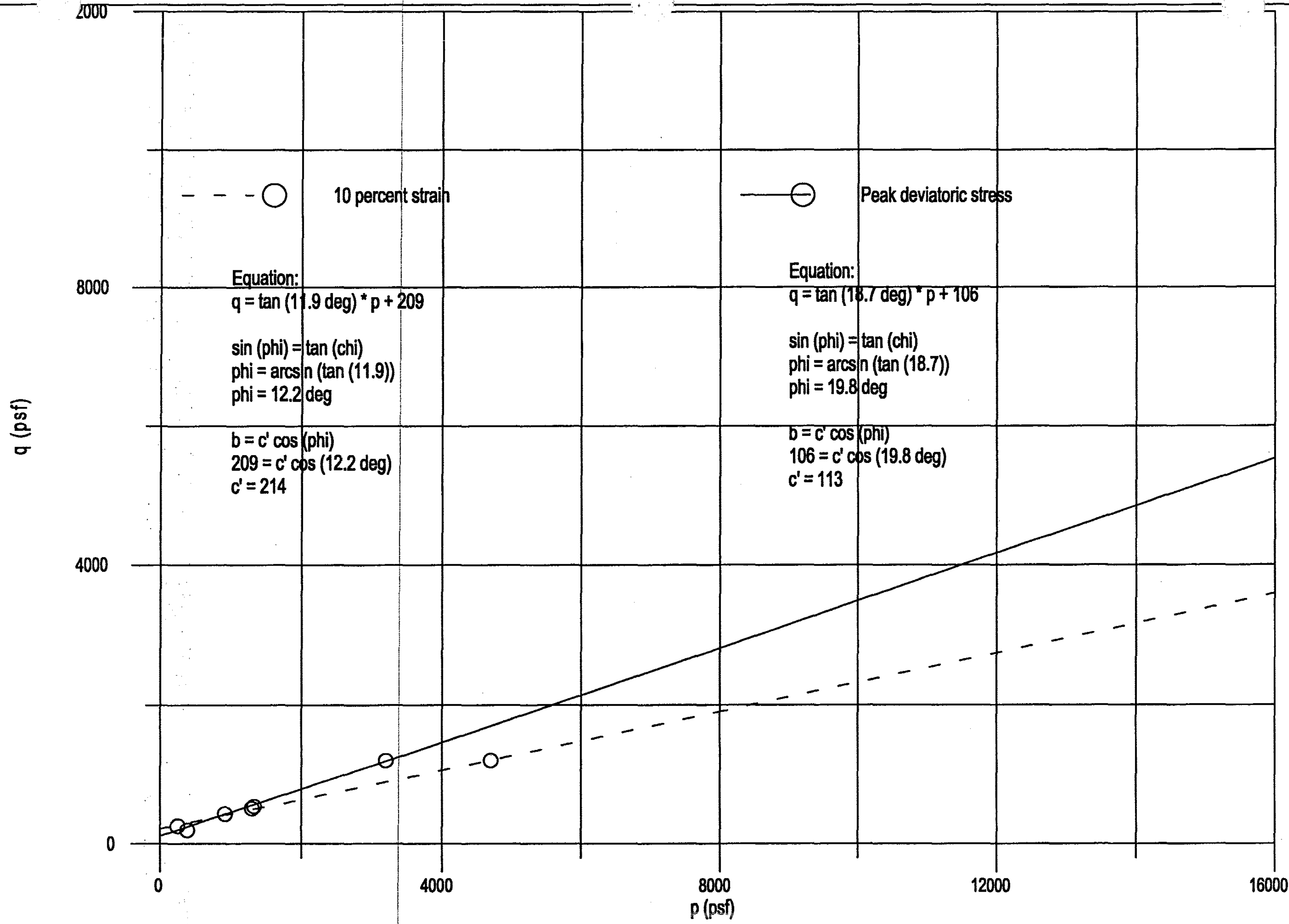
Equation:
q = tan (29.1 deg) * p

sin (phi) = tan (chi)
phi = arcsin (tan (29.1))
phi = 33.8 deg

Equation:
q = tan (28.0 deg) * p

sin (phi) = tan (chi)
phi = arcsin (tan (28.0))
phi = 32.2 deg

Project No. 95–973017NB.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	Shear Strength Parameters For Lean Clay Fill with Dry Density greater than 110 pcf	Figure 23
07/02/1999			
URS Greiner Woodward Clyde			



Project No. 95-973017NB.00 07/02/1999	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	Shear Strength Parameters For Fat Clay with Dry Density less than 80 pcf	Figure 24

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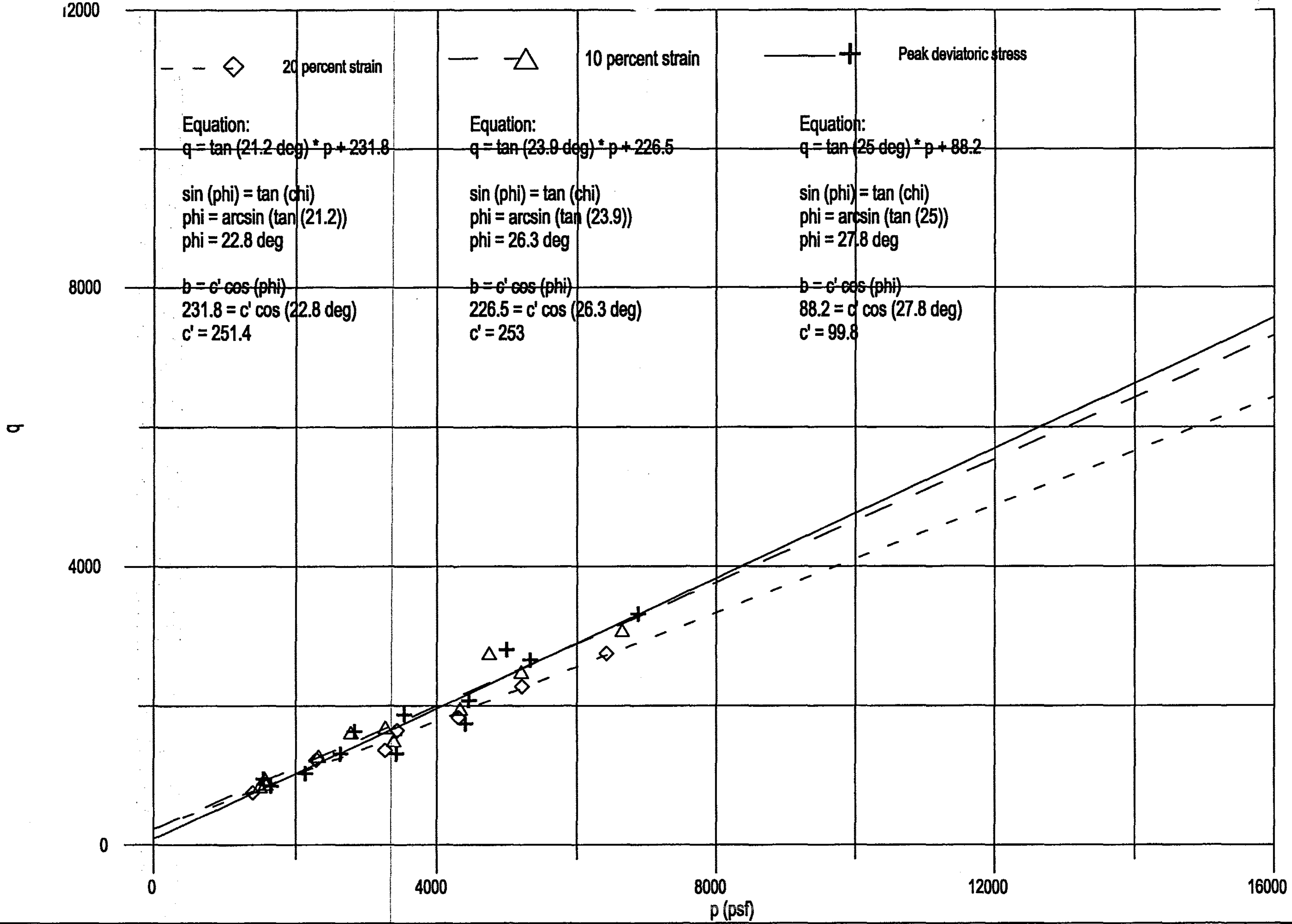

Equation:
q = tan (21.2 deg) * p + 231.8

sin (phi) = tan (chi)
phi = arcsin (tan (21.2))
phi = 22.8 deg

b = c' cos (phi)
231.8 = c' cos (22.8 deg)
c' = 251.4

Equation:
q = tan (23.9 deg) * p + 226.5

sin (phi) = tan (chi)
phi = arcsin (tan (23.9))
phi = 26.3 deg

b = c' cos (phi)
226.5 = c' cos (26.3 deg)
c' = 253

Equation:
q = tan (25 deg) * p + 88.2

sin (phi) = tan (chi)
phi = arcsin (tan (25))
phi = 27.8 deg

b = c' cos (phi)
88.2 = c' cos (27.8 deg)
c' = 99.8

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07/02/1999

LOWER GUADALUPE RIVER
FLOOD CONTROL PROJECT
Santa Clara, California

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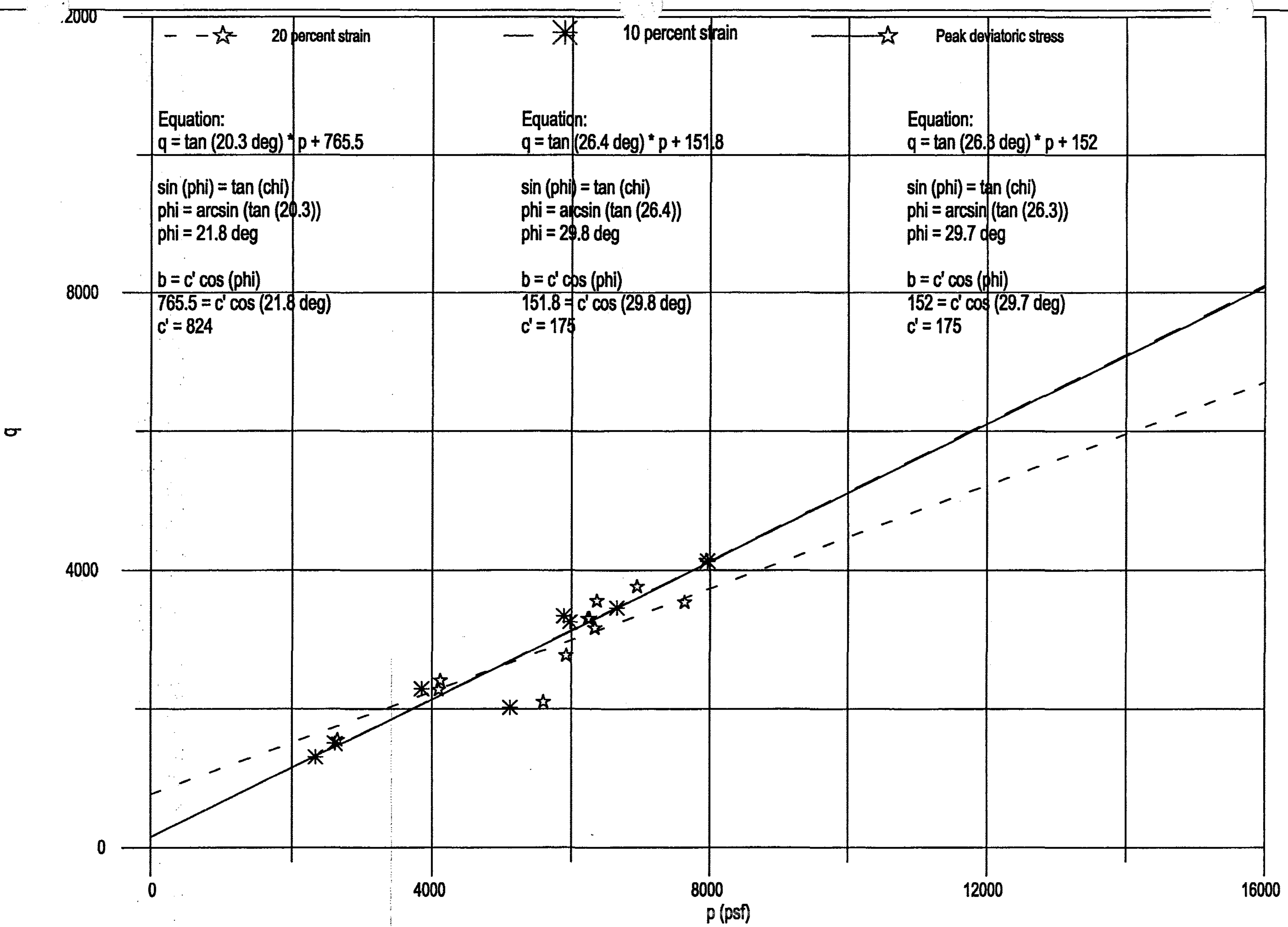
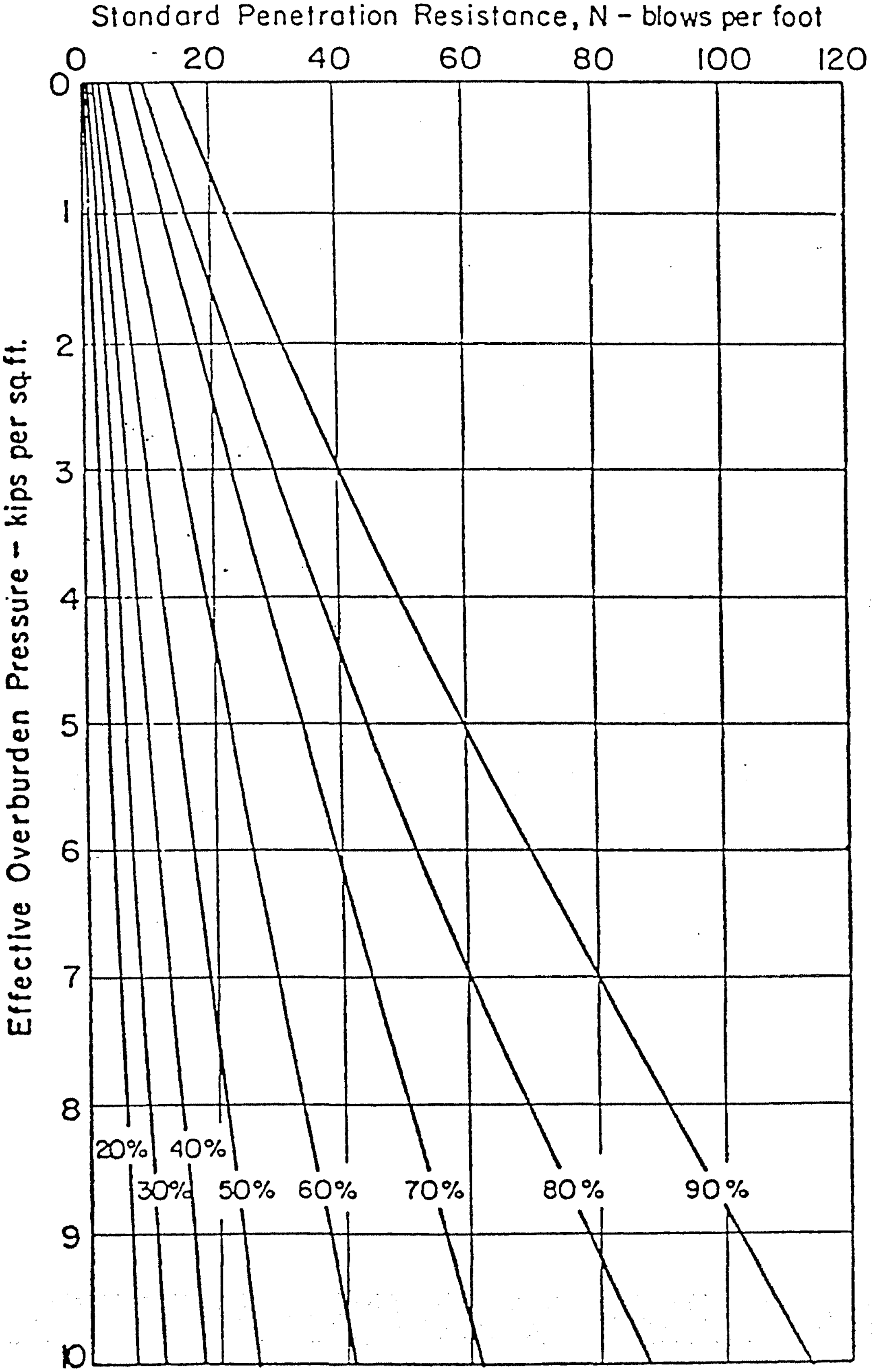
Shear Strength Parameters
For
Lean Clay with Dry Density between 80 pcf and 100 pcf

Figure
25



Project No. 95-973017NB.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	Shear Strength Parameters For Lean Clay with Dry Density between 100 pcf and 110 pcf	Figure 26
07/02/1999			
URS Greiner Woodward Clyde			



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LOWER GUADALUPE RIVER
FLOOD CONTROL PROJECT
Santa Clara, California

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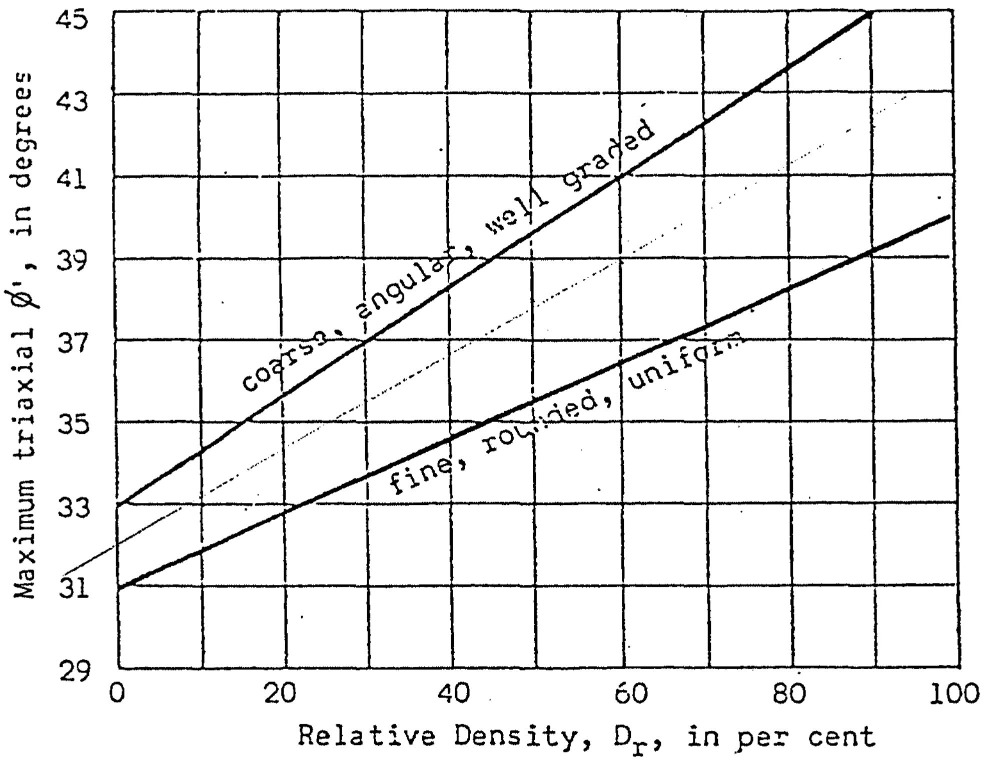
Relationship between Standard Penetration Resistance, Relative Density and Effective Overburden Pressure

(AFTER GIBBS AND HOLTZ, 1957)

Figure
27



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LOWER GUADALUPE RIVER
FLOOD CONTROL PROJECT
Santa Clara, California

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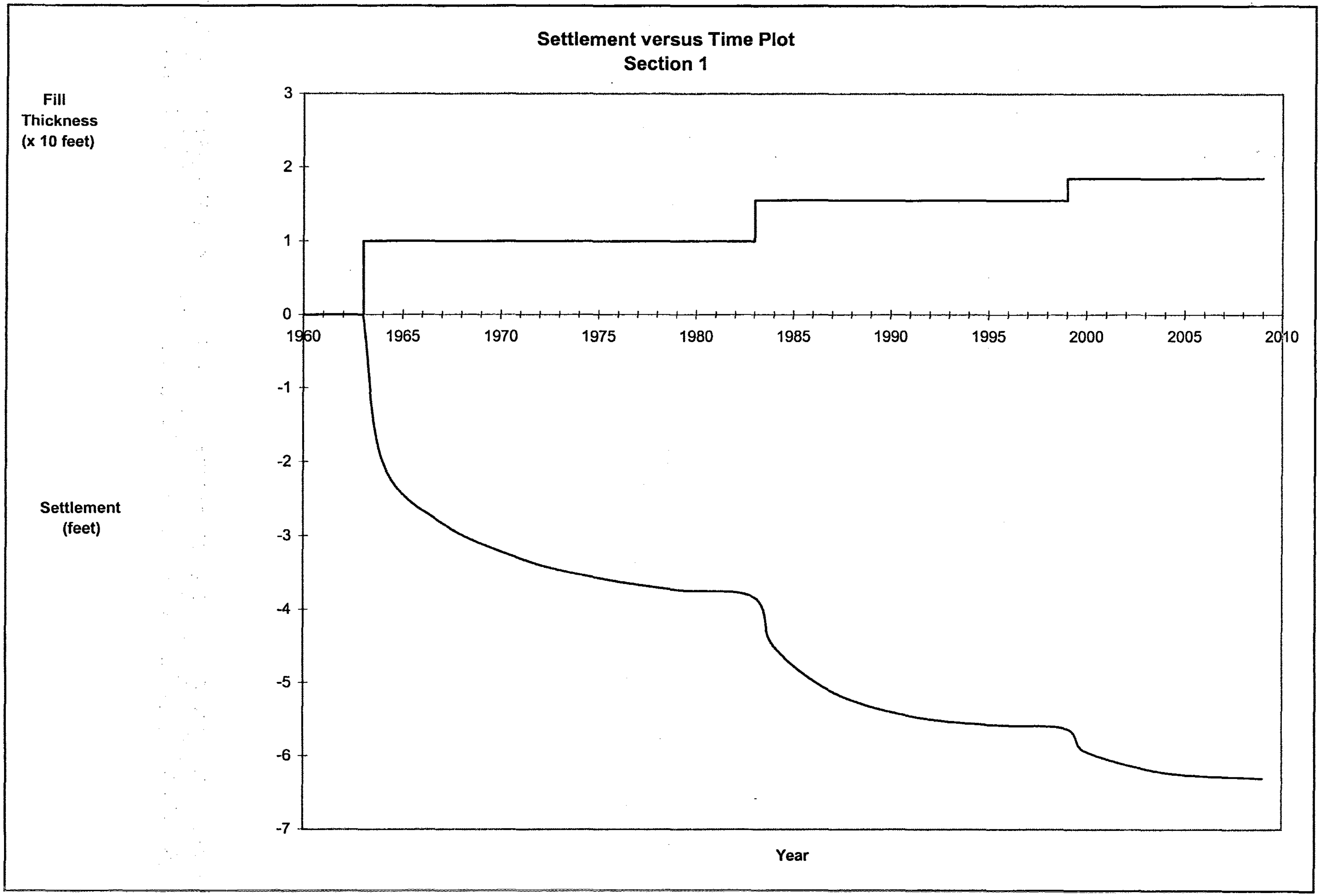
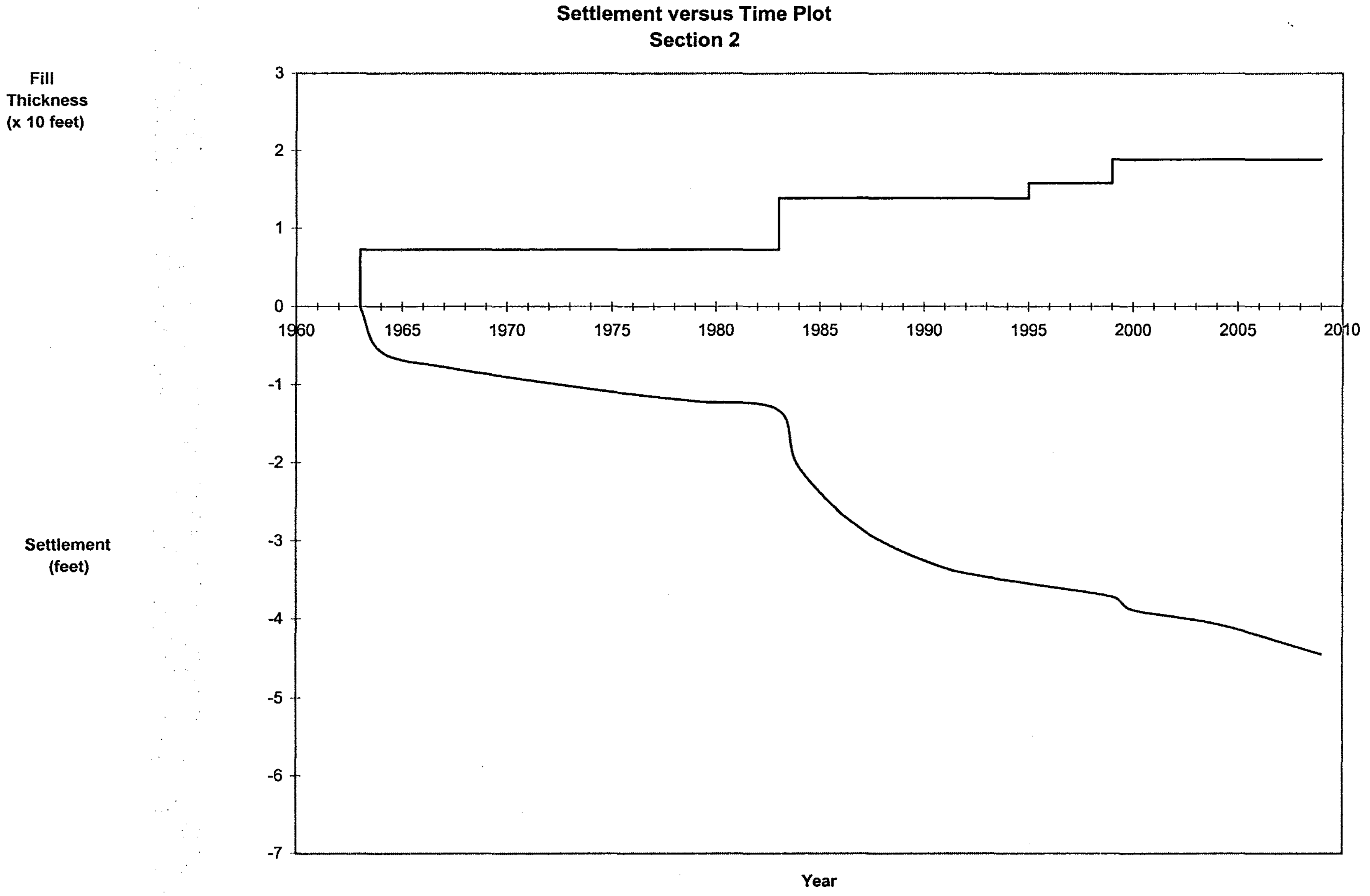
Approximate Correlation between the Effective Friction Angle and Relative Density for Quartz Sands
(AFTER SCHMERTMANN, 1975)

Figure
28

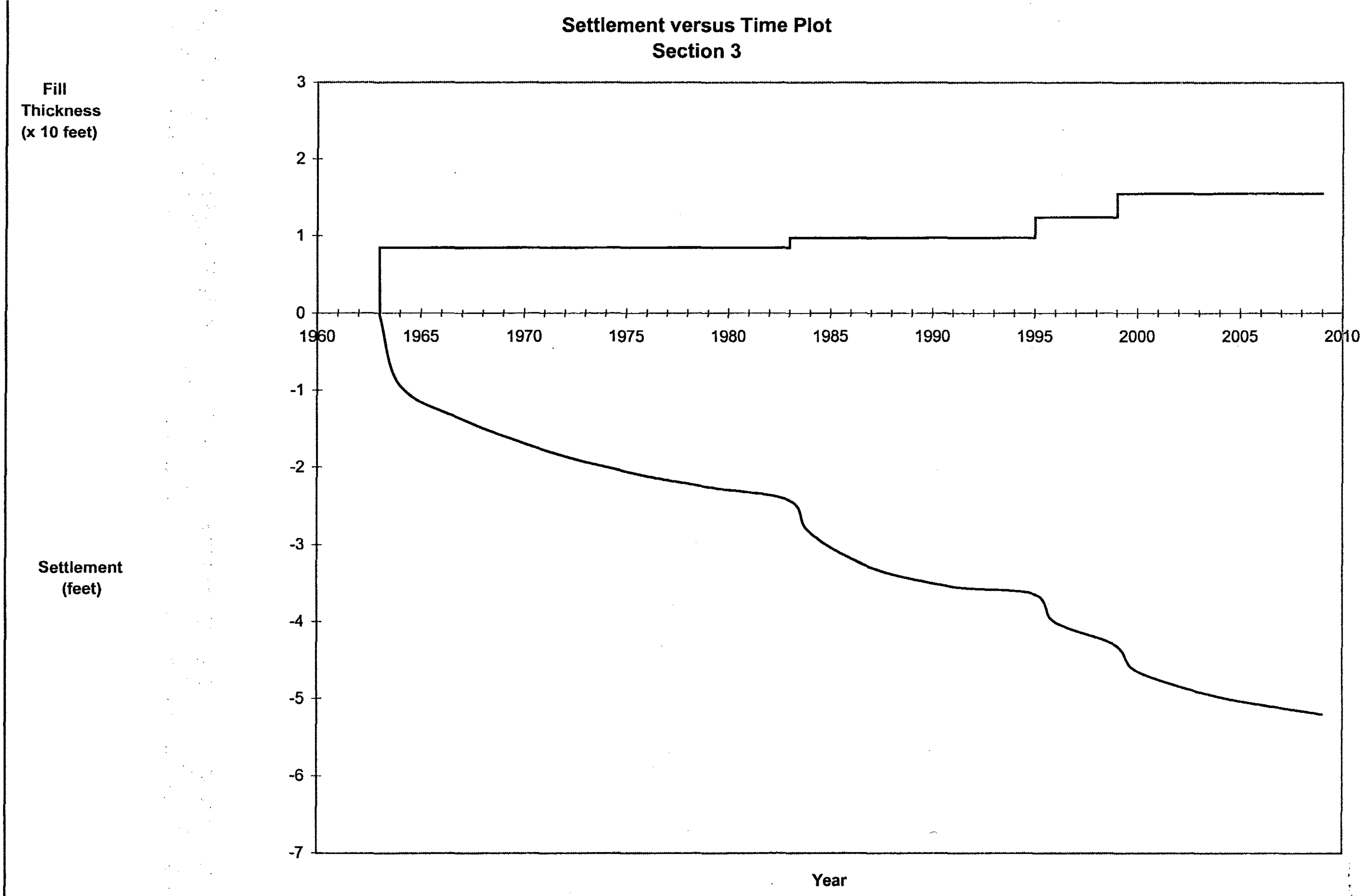


Project No. 95-973017NB.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	Estimated Time Rate of Settlement for Cross Section 1	Figure 29
06/30/1999			
URS Greiner Woodward Clyde			



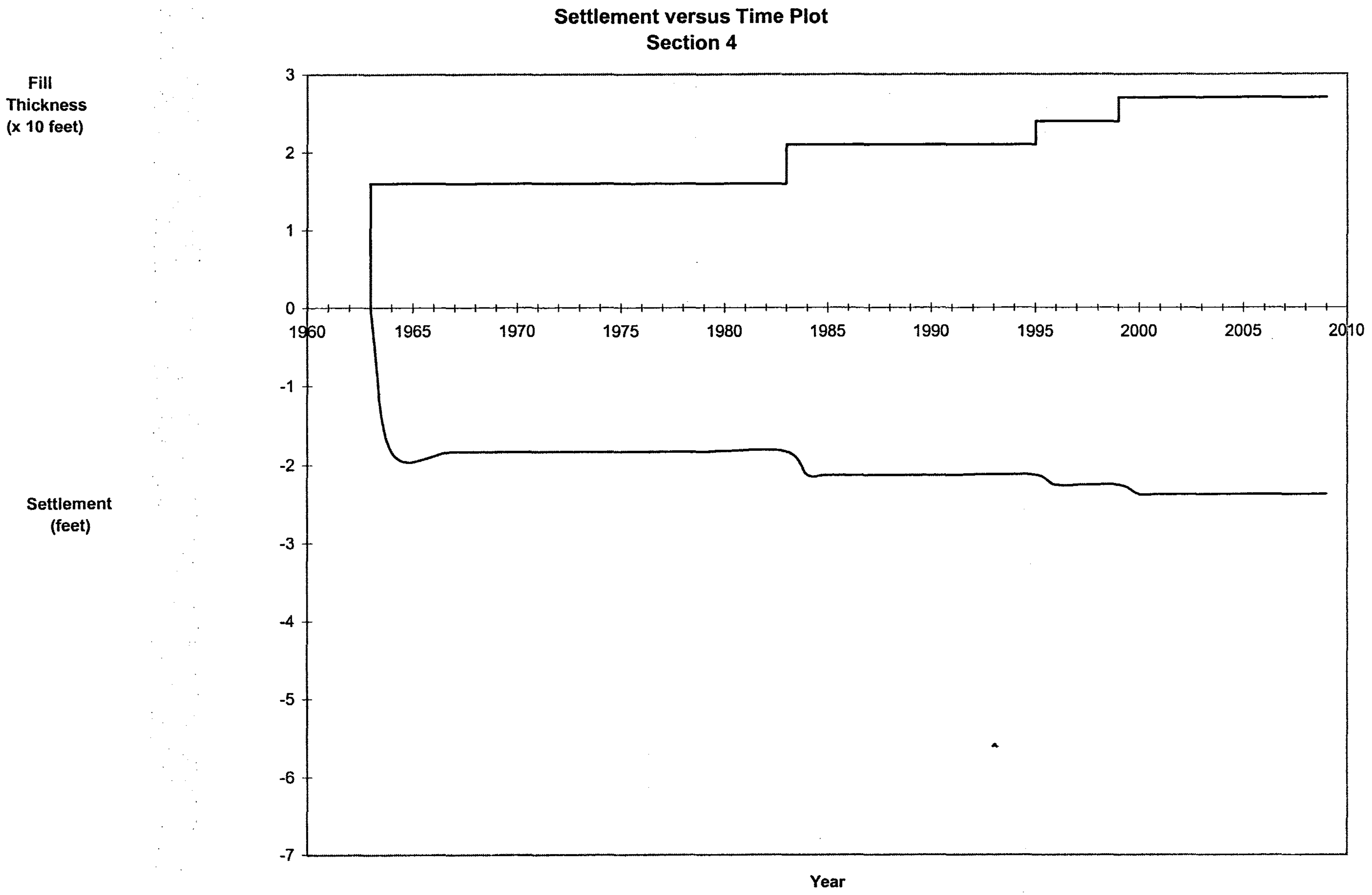
Project No. 95-973017NB.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	Estimated Time Rate of Settlement for Cross Section 2	Figure 30
06/30/1999			

URS Greiner Woodward Clyde

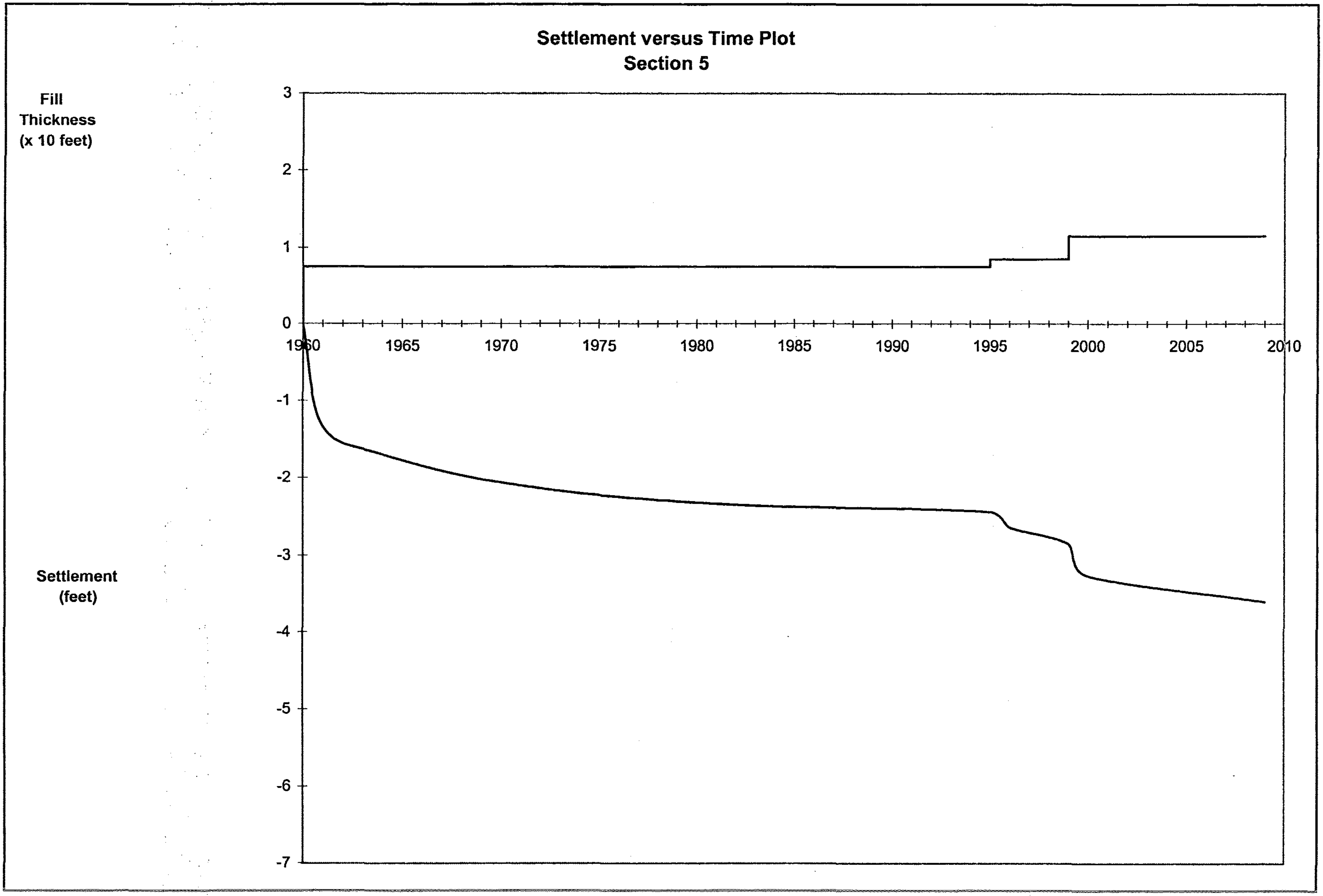


Project No. 95-973017NB.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	Estimated Time Rate of Settlement for Cross Section 3	Figure 31
06/30/1999			

URS Greiner Woodward Clyde



Project No. 95-973017NB.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	Estimated Time Rate of Settlement for Cross Section 4	Figure 32
06/30/1999			
URS Greiner Woodward Clyde			



Settlement versus Time Plot
Section 5

Project No. 95–973017NB.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California
06/30/1999	

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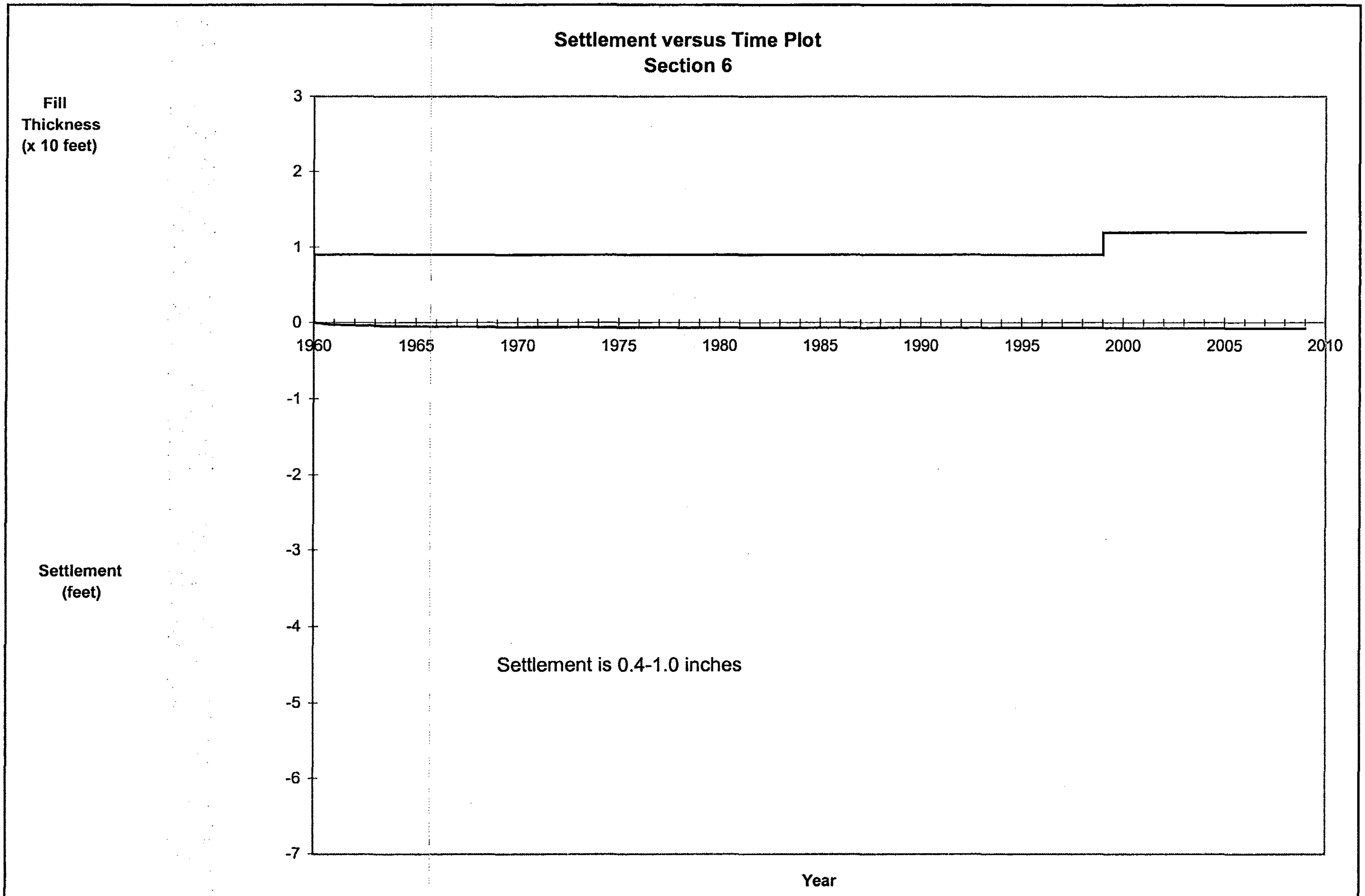
Estimated Time Rate of Settlement for Cross Section 5

Figure
33



Project No. 95-973017NB.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California
06/30/1999	

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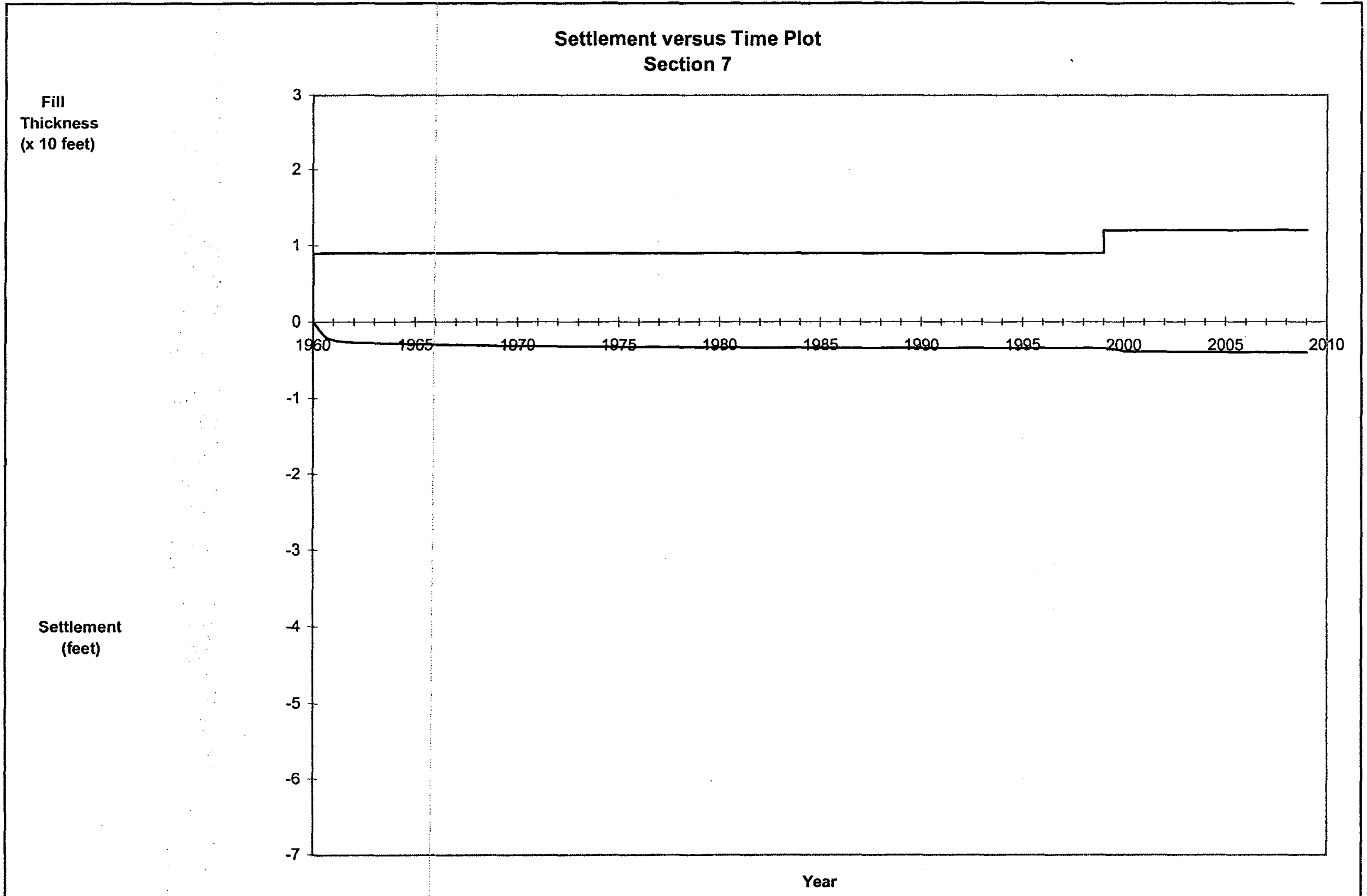
Estimated Time Rate of Settlement for Cross Section 6

Figure 34



Project No. 95-973017NB.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California
06/30/1999	

URS Greiner Woodward Clyde

Estimated Time Rate of Settlement for Cross Section 7

Figure 35

Coefficient of Permeability k in cm per sec (log scale)

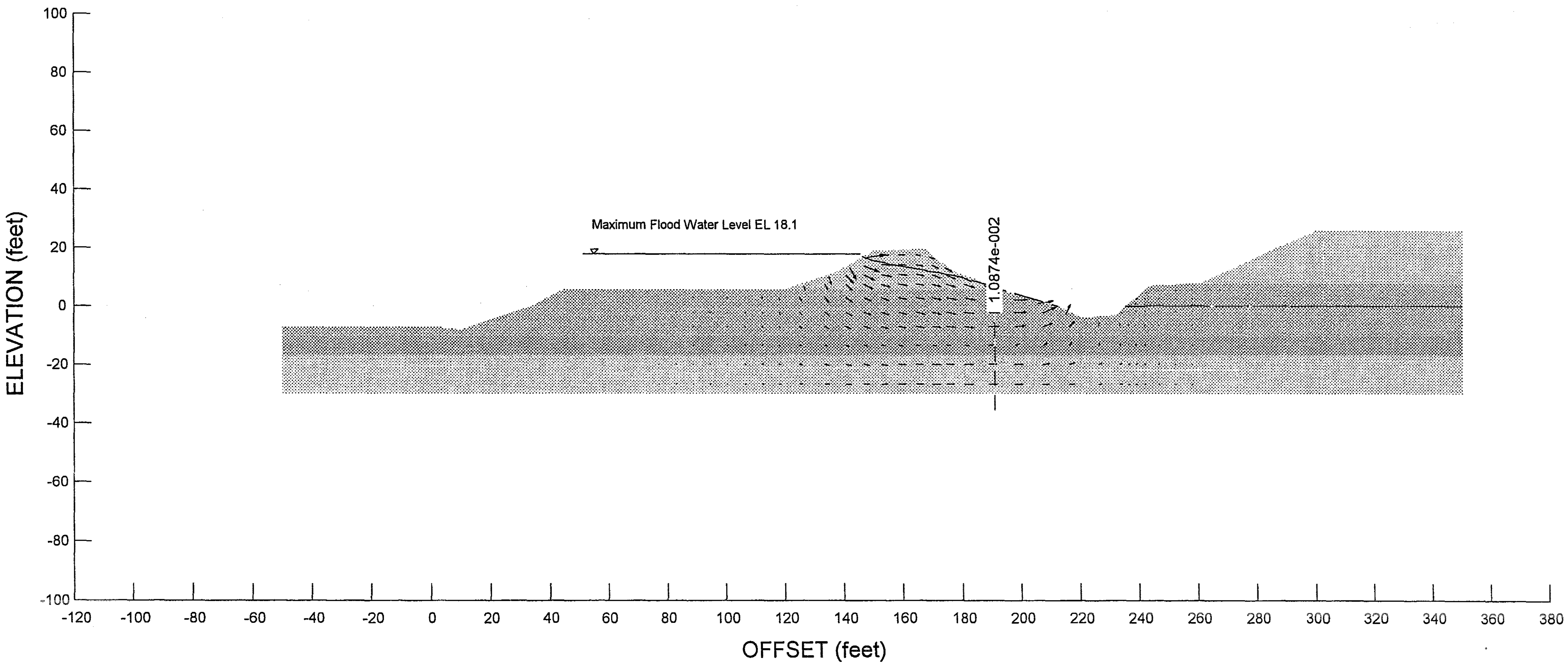
	10^2	10^1	1.0	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}	10^{-9}
Drainage	Good						Poor		Practically Impervious			
Soil types	Clean gravel		Clean sands, clean sand and gravel mixtures			Very fine sands, organic and inorganic silts, mixtures of sand silt and clay, glacial till, stratified clay deposits, etc.				"Impervious" soils, e.g., homogeneous clays below zone of weathering		
					"Impervious" soils modified by effects of vegetation and weathering							
Direct determination of k	Direct testing of soil in its original position—pumping tests. Reliable if properly conducted. Considerable experience required											
	Constant-head permeameter. Little experience required											
Indirect determination of k			Falling-head permeameter. Reliable. Little experience required			Falling-head permeameter. Unreliable. Much experience required			Falling-head permeameter. Fairly reliable. Considerable experience necessary			
	Computation from grain-size distribution. Applicable only to clean cohesionless sands and gravels									Computation based on results of consolidation tests. Reliable. Considerable experience required		

After A. Casagrande and R. E. Fadum

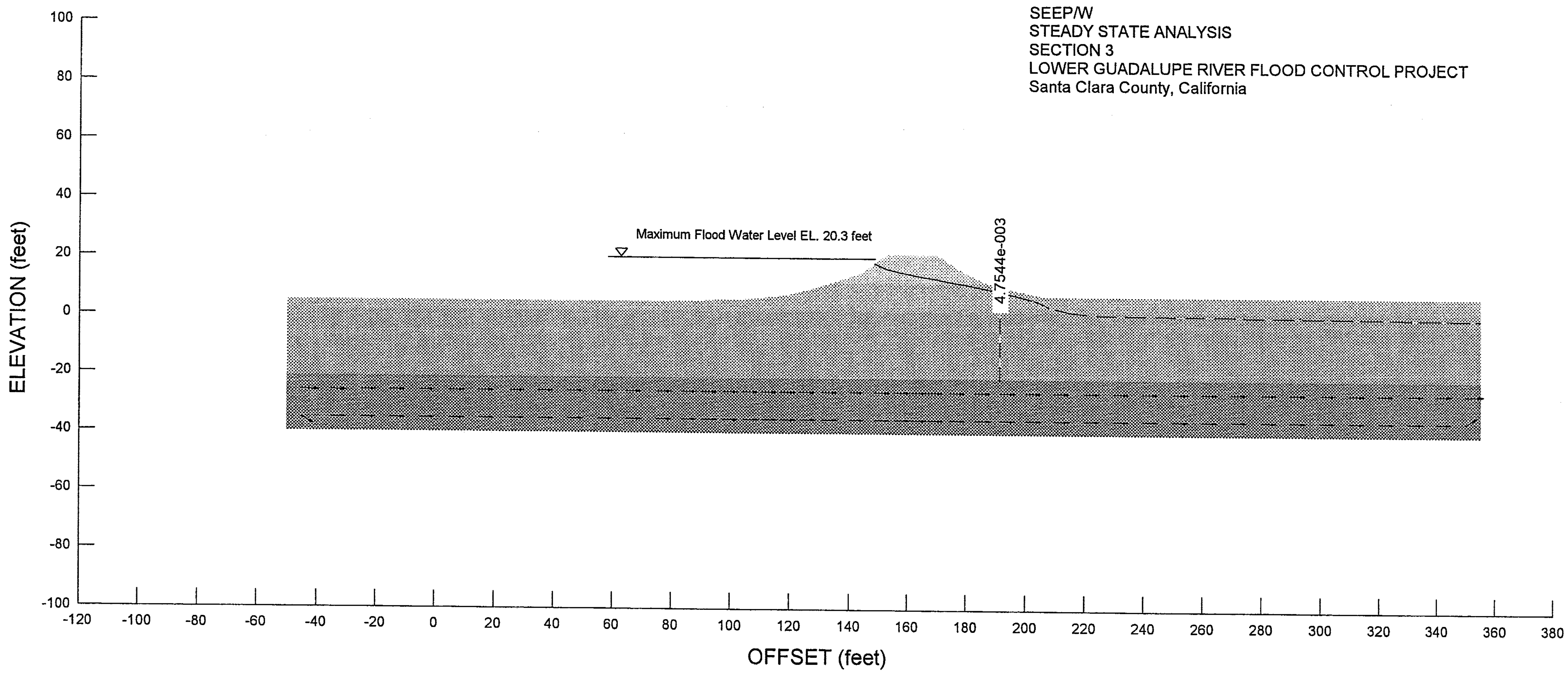
FIG. 2.8 Permeability and drainage characteristics of soils. (Table 6, p. 48, of *Soil Mechanics in Engineering Practice*, by K. Terzaghi and R. B. Peck, John Wiley and Sons, New York, 1948.)

Project No. 95-973017NB.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	Permeability and Drainage Characteristics of Soils	Figure 36
06/30/1999			
URS Greiner Woodward Clyde			

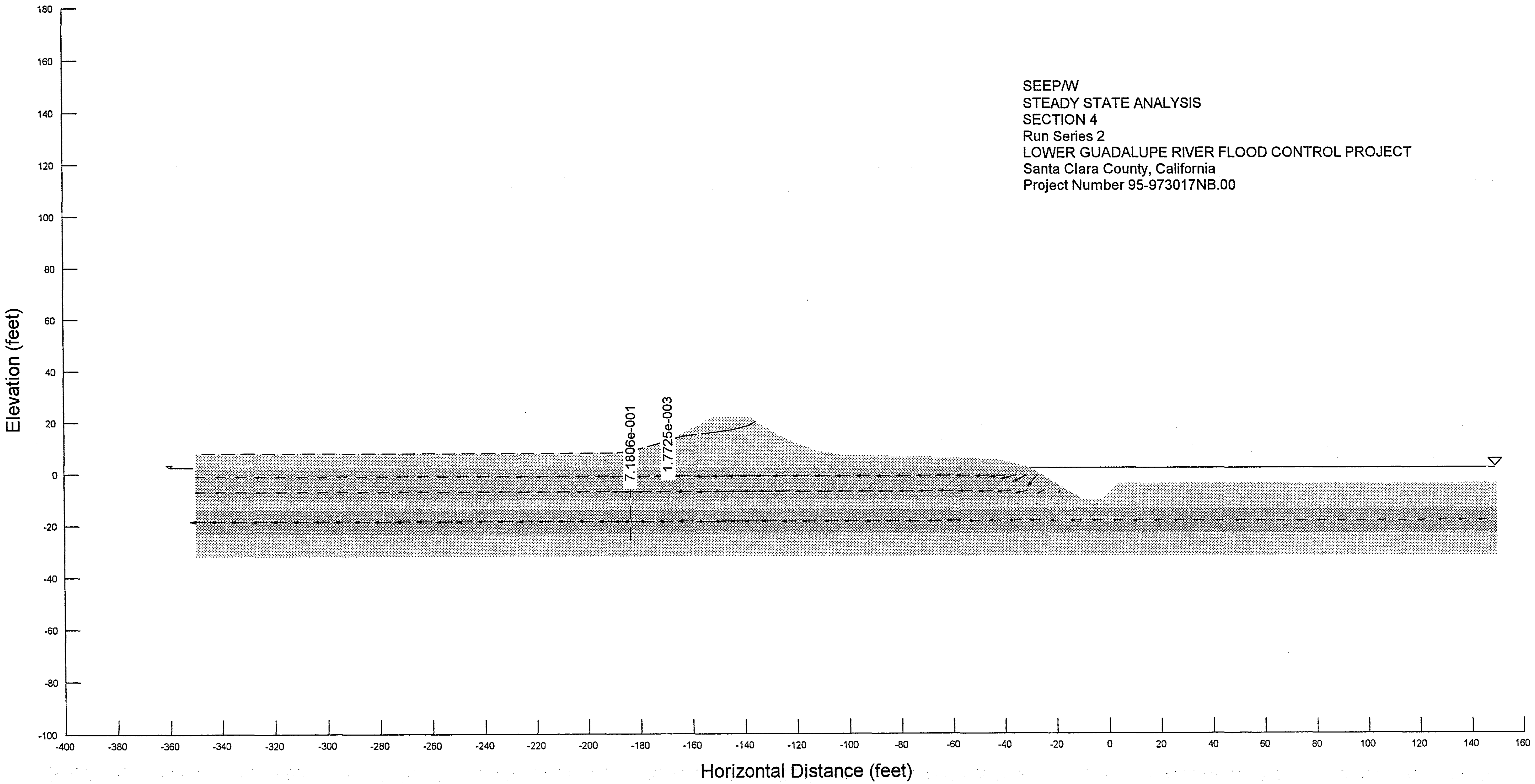
SEEP/W
STEADY STATE ANALYSIS
SECTION 2
Run Series 2
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT
Santa Clara County, California



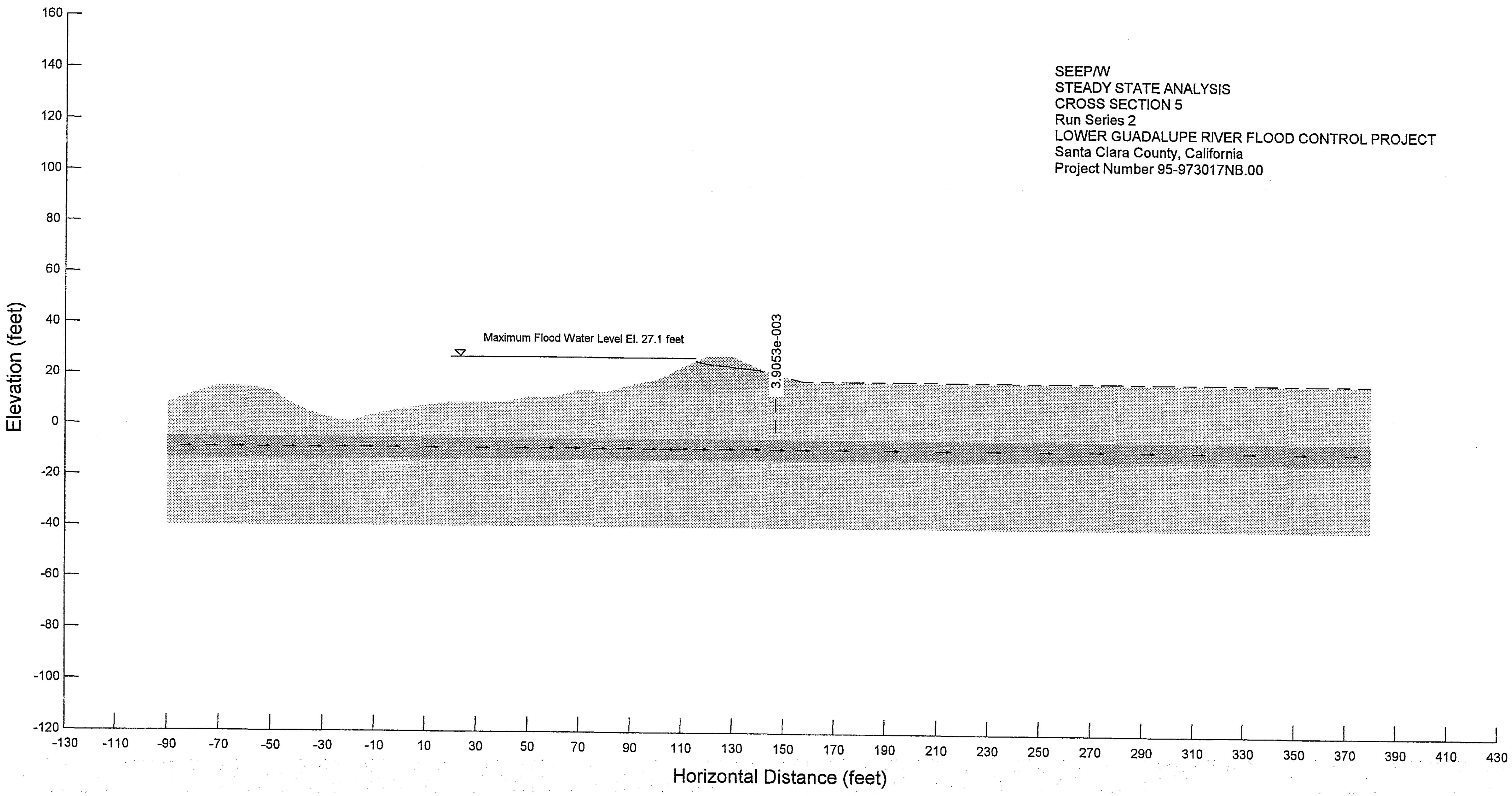
Project No. 95-9730717NB.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	STEADY STATE SEEPAGE ANALYSIS SECTION 2	Figure 37
06/30/1999			
URS Greiner Woodward Clyde			



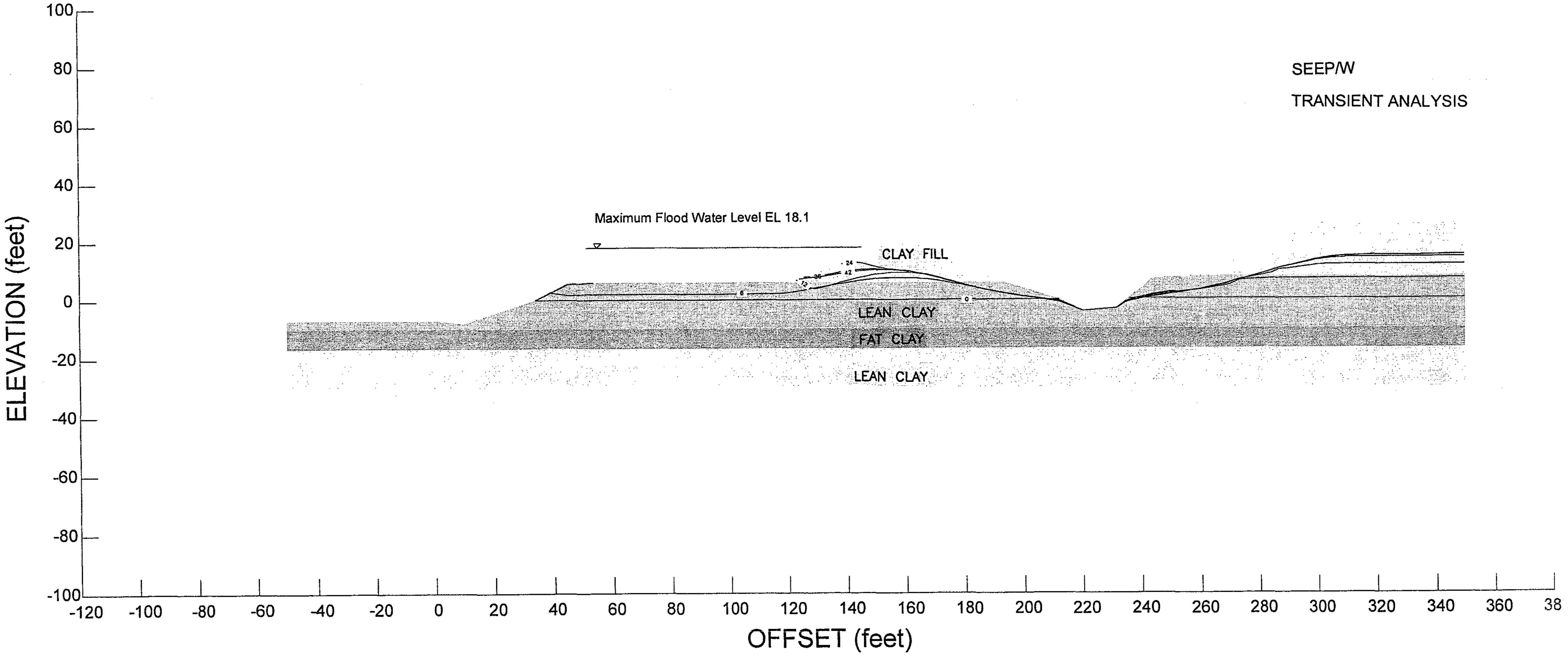
Project No. 95-9730717NB.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	STEADY STATE SEEPAGE ANALYSIS SECTION 3	Figure 38
06/30/1999			
URS Greiner Woodward Clyde			



Project No. 95-9730717NB.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	STEADY STATE SEEPAGE ANALYSIS SECTION 4	Figure 39
06/30/1999			
URS Greiner Woodward Clyde			



Project No. 95-9730717NB.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	STEADY STATE SEEPAGE ANALYSIS SECTION 5	Figure 40
06/30/1999			
URS Greiner Woodward Clyde			



LEGEND:

24 ——— PHREATIC SURFACE AT TIME STEP 24 (48 HOURS)

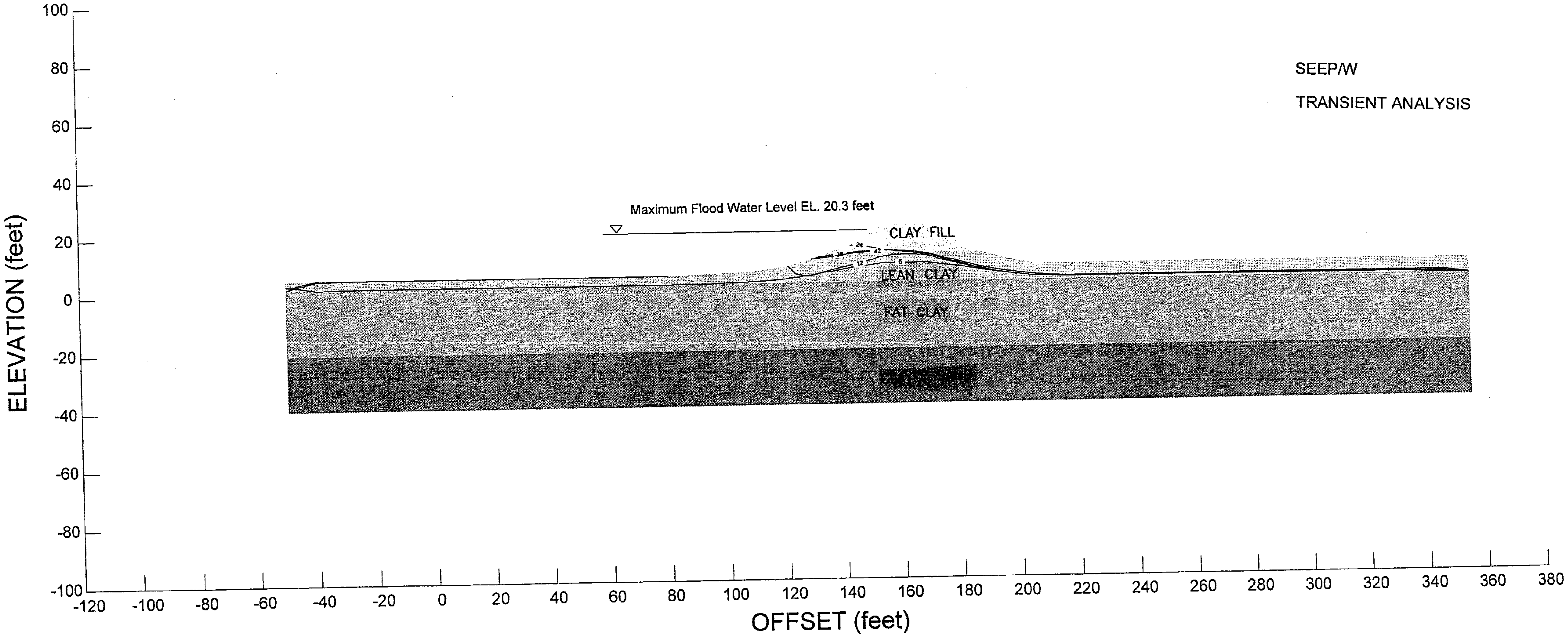
Project No.
95-9730717NB.00
06/30/1999

LOWER GUADALUPE RIVER
FLOOD CONTROL PROJECT
Santa Clara, California

URS Greiner Woodward Clyde

TIME DEPENDENT PHREATIC SURFACE
LINES, SECTION 2
(STA 8+610, WEST LINES)

Figure
41



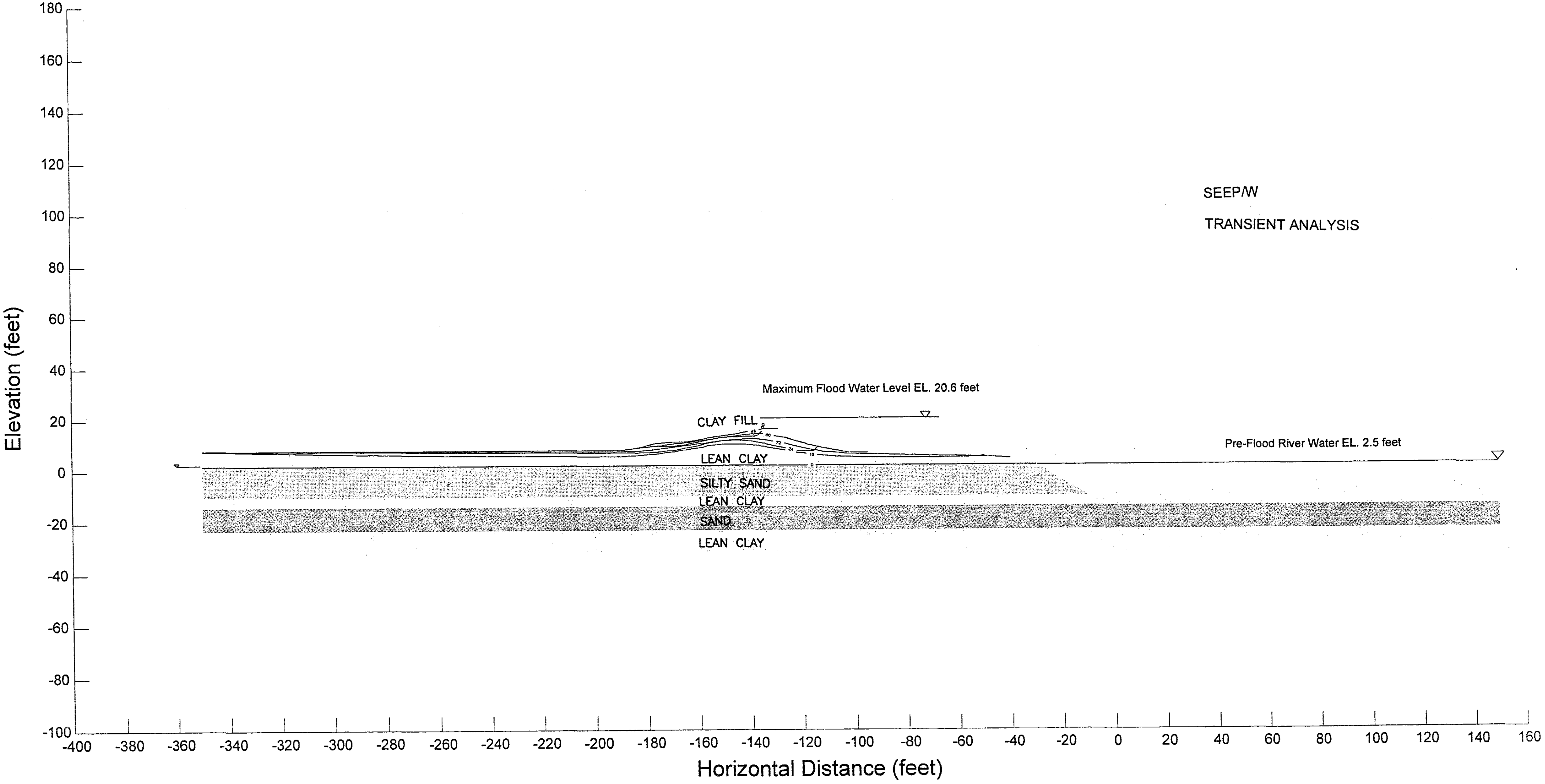
SEEP/W

TRANSIENT ANALYSIS

LEGEND:

24 ——— PHREATIC SURFACE AT TIME STEP 24 (48 HOURS)

Project No. 95-9730717NB.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	TIME DEPENDENT PHREATIC SURFACE LINES, SECTION 3 (STA 9+600, WEST LEVEE)	Figure 42
06/30/1999			
URS Greiner Woodward Clyde			



LEGEND:

24 ⁀ PHREATIC SURFACE AT TIME STEP 24 (48 HOURS)

Project No.
95-9730717NB.00

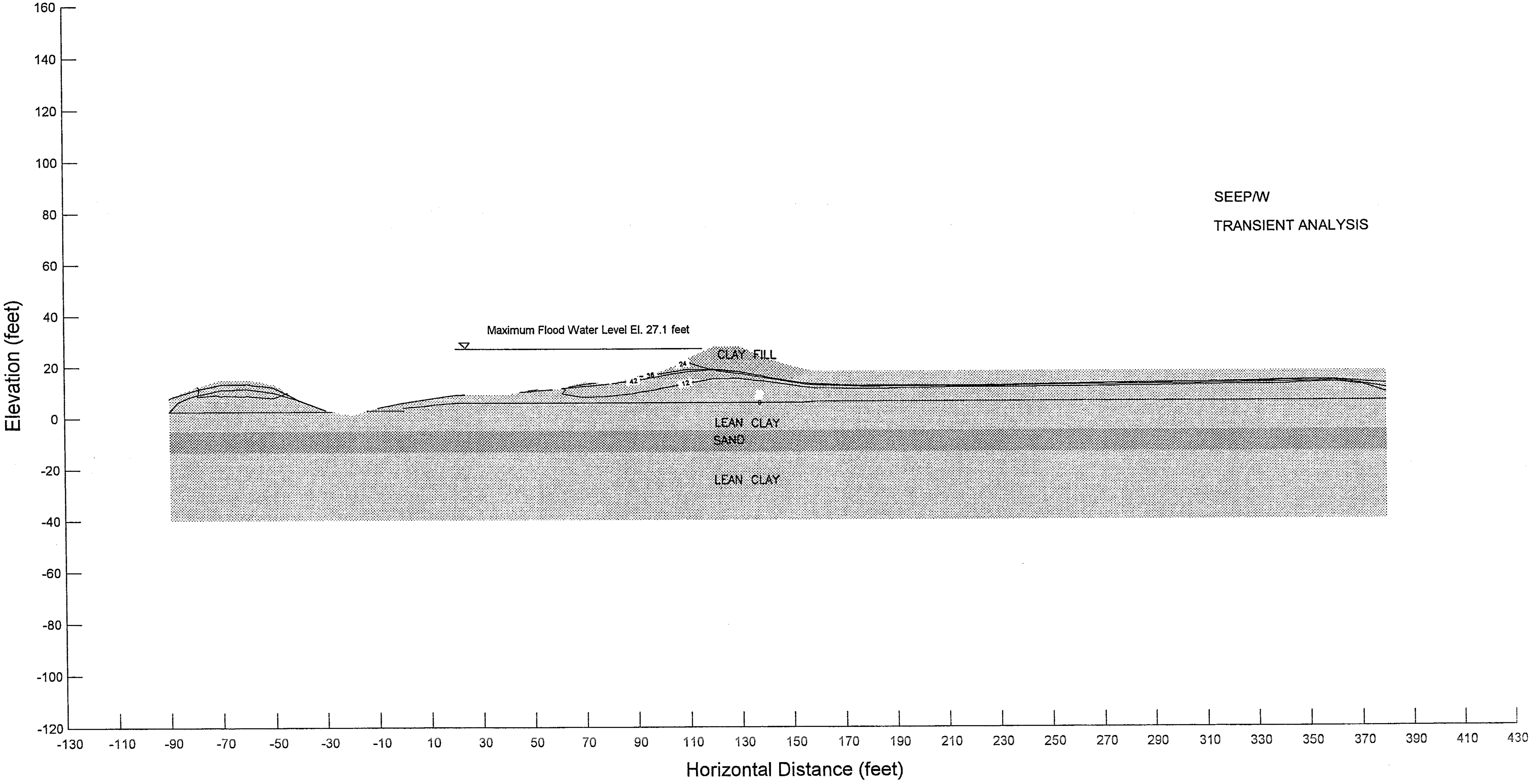
06/30/1999

LOWER GUADALUPE RIVER
FLOOD CONTROL PROJECT
Santa Clara, California

URS Greiner Woodward Clyde

TIME DEPENDENT PHREATIC SURFACE
LINES, SECTION 4
(STA 9+570, EAST LEVEE)

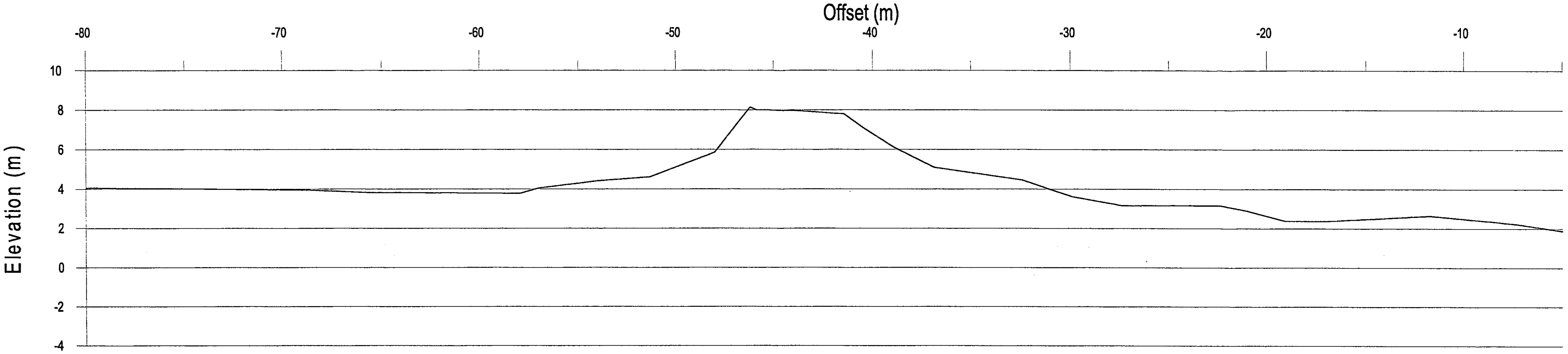
Figure
43



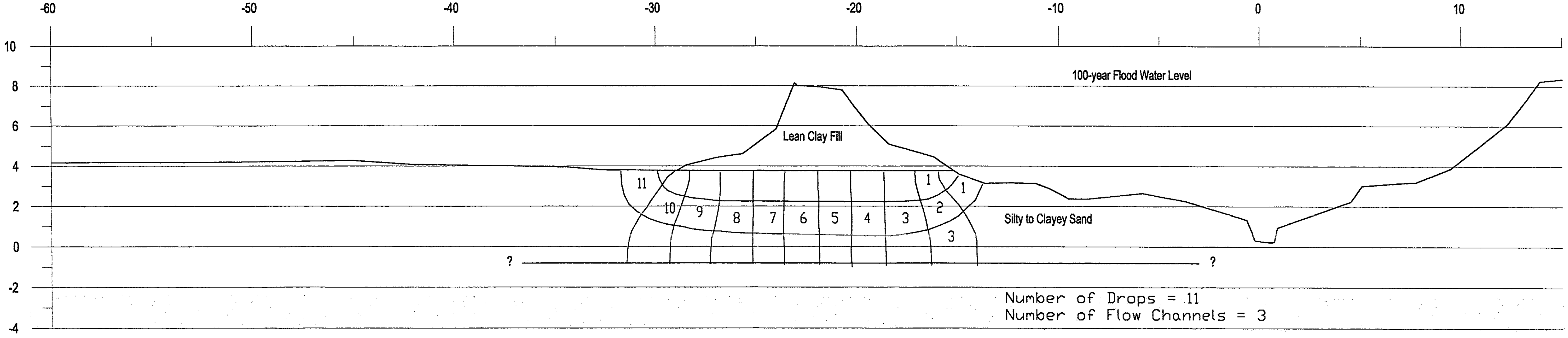
LEGEND:

24 ⌒ PHREATIC SURFACE AT TIME STEP 24 (48 HOURS)

Project No. 95-9730717NB.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	TIME DEPENDENT PHREATIC SURFACE LINES, SECTION 5 (STA 12+120, WEST LEVEE)	Figure 44
06/30/1999			
URS Greiner Woodward Clyde			



Scale 1 to 1 Cross-Section



Number of Drops = 11
Number of Flow Channels = 3

Project No.
95-9730717NB.00

06/30/1999

LOWER GUADALUPE RIVER
FLOOD CONTROL PROJECT
Santa Clara, California

URS Greiner Woodward Clyde

FLOW NET AT EAST LEVEE
NEAT BORING BORING EB-11
STATION 11+280, EAST LEVEE

Figure
45

Q:\CADDATA\CADI
ILES\973017NB\REPORT\MAIN\FLOWNET_F45

SOURCE: CHANNEL ALIGNMENT IS BASED ON AUTOCAD DRAWING PROVIDED BY SCVWD.

G:\GUAD004.DWG

REV	DESCRIPTION OF REVISION	BY	DATE
△	MOVED LOCATION OF SEVERAL BORINGS AND CPTS AND REVISED NUMBERS.	S.H.	3/19/98
△	MOVED WC-4 AND WB-5 PER CULTURAL RESOURCE RECOMMENDATION.	S.H.	4/9/98
△			
△			
△			
△			

URS Greiner Woodward Clyde

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WARNING
0 1/2 1
IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO SCALE

DESIGNED	S.H.
DRAWN	E.G.
CHECKED	S.H.
PEER REVIEWED	M.L.
PROJECT MANAGER	P.B.
DATE	7/15/99

Lower Guadalupe River Flood Control Project Santa Clara County, CA.

LOCATIONS OF POTENTIALLY LIQUEFIABLE LAYERS

REVISION △2

PROJECT 95-973017NB.00

DRAWING Fig. 46

SHEET OF

7/15/99

TEMP013